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REINFORCED CONCRETE POLES

By

R. D. COOMBS AND C. L. SLOCUM

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REINFORCED CONCRETE POLES.

By R. D. Coombs,
M. Am. Soc. C. E.,

and C. L. Slocum,
Asso. M. Am. Soc. C. E.

The increasing demands of the telephone, telegraph, light, and power companies, and the wide development of electric traction, together with the increased scarcity and cost of good timber poles, has compelled engineers to look for a suitable substitute possessing the desirable qualities of wooden poles, but without the necessity of continual maintenance and frequent renewals.

According to a report of the Forest Service, United States Department of Agriculture, the telegraph and telephone companies purchase about two-thirds of the total number of timber poles used each year. The remainder may be credited to the steam and electric railroads, and the electric light and power companies. The total number of timber poles over 20 feet in length purchased in 1906 was reported as 3,574,666, and their value, at the point of purchase, as $9,471,171, or an average of $2.65 per pole. In Table 1 are shown the number and average value, at the point of purchase, for varying lengths of the five leading varieties of timber. Other varieties, and the sawed poles of the varieties given, are omitted from the table, since their combined number is relatively small.

By far the greater number of poles are cedar and chestnut, and, as the former grow in the Lake States, Maine, northern New York, and Idaho, and the latter in Pennsylvania, Maryland, Virginia, and West Virginia, the item of freight to be added to the tabular value may be a considerable factor of the final cost. The rapid increase in the cost of timber, which, as shown by Table 1, is still further increased for long lengths, and the deterioration of unpreserved timber, have forced purchasers to consider the use of other methods and materials.

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TABLE 1.—ROUND POLES (1906).*

	Length 20 to 25 Feet.		Length 26 to 30 Feet.		Length 31 to 35 Feet.		Length 36 to 40 Feet.		Length 41 Feet and Over.		Totals.	
	Number.	Average Value.	Number.	Average Value.	Number.	Average Value.	Number.	Average Value.	Number.	Average Value.	Number.	Average Value.
Cedar ...	1,305,148	$1.19	408,139	$3.22	262,739	$4.94	123,391	$6.17	70,452	$9.08	2,169,869	$2.57
Chestnut.	404,877	$1.42	265,315	$2.52	184,028	$3.35	75,108	$4.64	57,975	$7.08	987,303	$2.65
Pine.....	77,730	$1.68	30,520	$3.18	25,914	$4.84	15,828	$5.13	12,609	$12.41	162,601	$3.63
Cypress .	27,041	$1.09	40,263	$1.24	22,700	$3.04	14,101	$4.42	7,187	$6.28	111,292	$2.30
Juniper ..	24,063	$1.62	12,003	$2.70	10,638	$3.68	4,113	$4.09	6,247	$5.76	57,064	$2.86
Totals .	1,838,859	$1.27	756,240	$2.86	506,019	$4.24	232,541	$5.50	154,470	$8.33	3,488,129	$2.60

In its function as a carrier of wires a pole resists downward, lateral, and, to some degree, torsional forces. A little strength against compression, a superior resiliency, and a long life in a variable climate and soil, are the chief requirements of a good pole.

Until recently wooden poles have been so cheap that the advisability of using a wood preservative to delay decay has not been widely or seriously considered, and because the expense of treating the entire pole exceeded the additional benefit or life attained. In addition, many poles have to be renewed not only on account of decay, but because poles of larger capacity are required. The future demands upon an installation cannot always be foretold with accuracy. The duration of the useful life of a timber pole, in contact with the soil, depends in part upon the chemical action of the ingredients of the earth and upon its ability to resist local insect life. Disintegration will, therefore, advance more rapidly in some soils than in others, but in general the use of native timber for local use will be found advisable. The zone of decay at the ground-line is produced by alternate wetting and drying, inducing a condition of decay which frays away the body of the pole until this critical section is so emaciated that it will no longer sustain its load. In the dry season this decayed portion is much in the nature of dry tinder, and if the pole is located on a grassy right of way, grass fires char away still more of the critical section. An application of coal-tar to this portion of the pole, while proper in desert localities, would promote the early destruction of the pole in places subject to running grass fires.

Preservative treatment and the consequent use of inferior grades of timber will no doubt afford temporary relief, but it is entirely

* United States Forest Service, Circular No. 137.

probable that within the next decade some form of artificial pole will be able to compete in first cost with the wooden poles then available. In case it is necessary to use long, heavy poles, or if the character of line is such that safety and permanence are prime requisites, it will frequently be economical to use reinforced concrete poles.

In addition to the timber poles there are used each year a relatively small though increasing number of metal poles. Steel poles or towers are coming into more general use for power transmission lines, particularly as applied to long spans or high poles. Until recently such steel towers have been built of the lightest sections, often from $\frac{1}{8}$ to $\frac{1}{4}$ inch in thickness, and thus requiring great care in handling and frequent painting. As a rule, structures exposed to the elements are not given frequent attention, and are only repainted after oxidation has occurred to a marked extent.

During the last few years steel transmission line poles of substantial construction, using sections whose thickness and length ratios are in accord with the best modern practice, have been built by several of the large eastern railroads. Some of the lines referred to carry a large number of heavy wires, and for this and other reasons were not well adapted to the use of wooden poles.

Since steel is comparatively expensive and requires maintenance to prevent corrosion, considerable attention has been given to the use of reinforced concrete poles for both telegraph and transmission line construction.

When steel is embedded in well-made concrete its preservation is perfect, and the life of a reinforced monolith is practically indefinite. If designed and built with the same attention now given other materials, reinforced concrete poles should attain the necessary strength and give satisfactory service. As in the case of steel poles, they can be spaced greater distances apart than is economically possible with wooden poles, and in their fire-resisting qualities are at least equal to steel poles. This latter feature will become of increased importance with the spread of modern requirements for fire protection.

Concrete poles are of a pleasing gray color and are readily modified in outline, or in the treatment of the base, to suit the locality in which they may be situated.

By the insertion of pipes, or the formation of an axial passage in the concrete, wires may be carried from the pole tops to the ground, and thence in any desired direction, and are thus entirely protected at little additional cost.

In damp climates, or in localities where wooden poles are subject to attack by fungi, or insects, concrete poles have a longer life than either steel or timber.

On long or important transmission lines where reliability of service is of great value, it may be conceded that the additional expense of a material superior to timber will often be warranted.

Owing to the natural taper of the timber, it is frequently the case that the weakest section of a timber pole is at some point above the ground level. Therefore there is an excess of material in the butt, which may be considered wasted, except in so far as this surplus timber is useful in resisting decay. A reinforced concrete pole may be given any desired taper and need have no excess of improperly placed material.

The character of service required of line poles is not that of a column, as might at first be supposed, but of a cantilever beam. Further, in order to reduce the stresses in the pole under certain conditions of loading, it becomes necessary for the pole to deflect in the direction of the line, and therefore a certain elasticity is desirable in the material.

If we may judge by the kind of handling which concrete piles successfully withstand, it would seem entirely probable that concrete poles will survive any shocks incident to ordinary service. When subjected to an overload or accidental shock, a timber pole will bend and in some cases survive; but failure, when it does occur, is usually complete, and the pole falls. Concrete poles, on the contrary, while without the elasticity of timber, do not fail by breaking off, but are held by the reinforcement from falling to the ground. Tests also show that a reasonable amount of bending (sufficient for the balancing of stresses in the wires) can occur without apparent injury to the pole.

The chief cause of skepticism heretofore has been the fear that such long, slender members would not be able to withstand, without cracking, the bending stresses and measurable deflections of a pole line. If the poles are properly designed, cracks due to partial failure will not occur. Hair cracks are of infrequent occurrence, microscopic in character, and experience has shown that they will not admit moisture in sufficient quantity to injure a reinforced concrete structure.

In view of the various successful installations in this country and in Europe, and assuming that proper unit stresses are used in designing, and the necessary care taken to obtain a dense mixture

and a good surface finish, the writers do not believe that there need be any apprehension of injury due to cracks.

The location of pole lines is not always well adapted to the convenient transportation of materials, and, as the erection of such lines is frequently done by hand or with light rigging, it is not desirable that poles should be of great weight. The greater weight of concrete poles, rendering their shipment a matter of increased expense, as compared with timber, and the possibility of injury in handling to the site, introduces a question as to the relative advantage of manufacture at the site or at distant yards. In many cases it will be found advantageous to manufacture poles at one or more favorably located points in order to avoid the transportation of raw materials, forms, housing, men, water, etc., and because it is not always possible to obtain space for manufacture immediately adjacent to the site. On the other hand, certain conditions of inaccessibility will make it desirable to haul raw materials to the site, rather than to attempt the more difficult handling of long monolithic poles. The investigation reduces to the availability of the material for the service required and the relative cost. The matter of cost is complicated by the locality of manufacture and the cost of erection, so that at the present time each installation must be judged separately, and the real question at issue is one of availability. It may be noted in passing that, in a number of instances, reinforced concrete poles have been installed at a lower cost than steel or wood.

History of the Development of Concrete Poles.

The earliest concrete poles in America were designed and erected on the Isthmus of Panama by Col. G. M. Totten, Chief Engineer of the Panama Railroad Company, about 1856. Concrete was used on account of the ravages of insects. These poles were about 12 feet long, circular in section, having a 6- to 8-inch top and 12- to 15-inch base. The wires were carried on iron bracket cross-arms, fastened to the tops of the poles by wrought-iron bands. The proportions of the concrete are not now obtainable. The first poles were entirely of concrete, but since they were not capable of withstanding lateral strains, they were replaced by poles reinforced with a 3 by 3-inch wooden core. This latter construction was also a failure, because the wooden cores swelled and cracked the concrete, and both types of poles were abandoned, so that, in 1888, there were only about twenty of the original installation standing.

About 1900 the practice of using concrete bases around the decayed butts of wooden poles became quite common. It is alleged that these poles are better than new ones, and that a saving of 35 to 55 per cent. is made by their use in reconstruction.

The first use of reinforced concrete poles in Europe is perhaps uncertain, but a French engineer, M. Hennebique, was probably the originator of this form of construction. The trolley poles built by him in 1896, for the Le Mans Tramway Company, in France, are in use to-day. These poles are solid, circular in section, and reinforced with small round rods and transverse wires.

In 1900, M Porcheddu made the test given below, upon a Hennebique pole, for the Societa Anomina di Elettricita Alba Italia, of Bologne, Italy. Some of these poles are in use in a tramway line between Borgone and Russoleno. They are about 35 feet long, having a 15-inch base and a 7-inch top, and are of solid square cross-section. Small round rods were used as reinforcement.

Diameter at top	6.3 inches
Diameter at bottom	13.8 inches
Total length of pole	35.0 feet
Length above ground	29.5 feet
Distance of load above ground	27.7 feet
Distance of load above point of failure	7.0 feet

In Table 2 are given the elastic and also the permanent deformations for increasing test loads; the point of application of the load being about one foot below the top of the pole.

TABLE 2.

Pull in Pounds.	Deformation in Inches.	Permanent Deformation.
463	1.06	0.00
926	3.34	0.00
1300	5.50	0.71
1521	7.27	0.71
1962	10.61	0.71
2182	14.34	1.57
2205	14.93	2.75
2866	15.33	2.75
4012	28.30	2.75
4410	. . .	Rupture.

Fig. 1 represents graphically the behavior of this pole under various loads.

M. Porcheddu also tested a design of his own, and a number of poles of this type were afterward placed in a tramway line. This pole

was about 35 feet in length, square in cross-section, 7 inches at the top and 15 inches at the bottom, solid, and reinforced with small smooth rods. A pull of 4000 pounds, at the top, gave a maximum

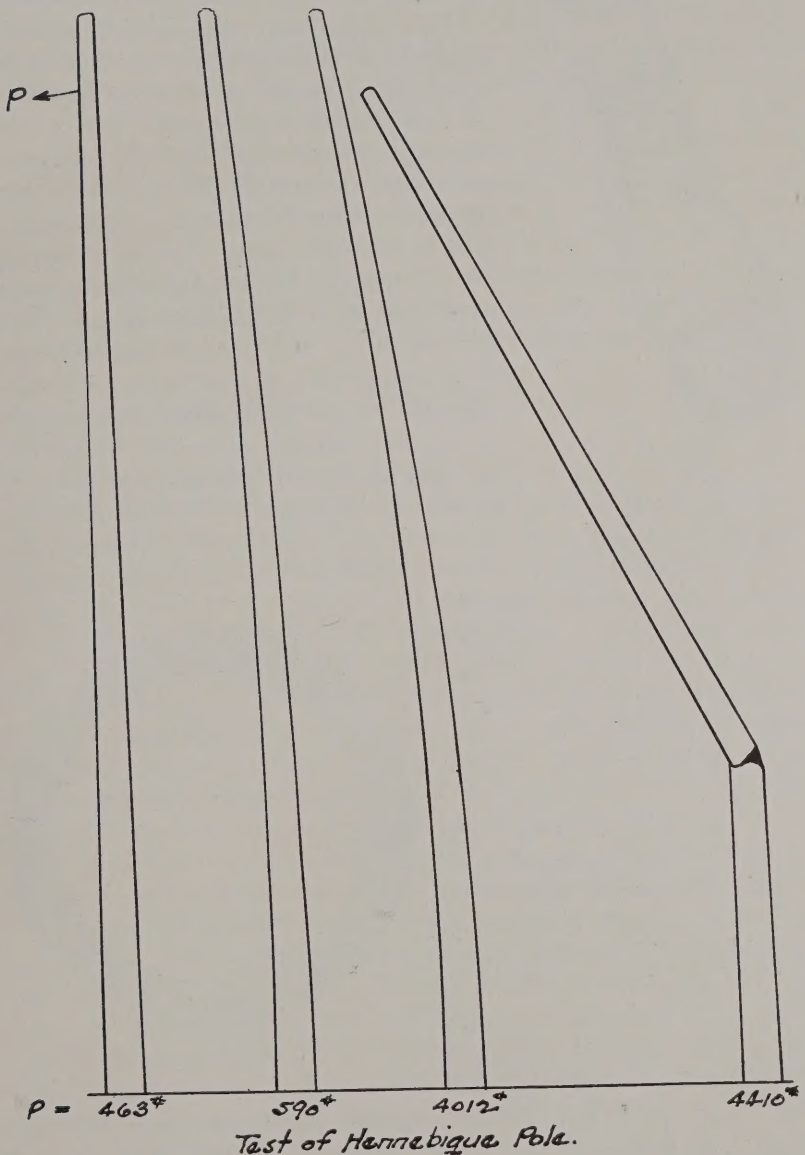


Fig. 1.

deflection of 2 feet 6 inches, the pole returning to within 3 inches of the normal position. This pole safely withstood 4500 pounds and broke at 4700 pounds, but was held by its reinforcement from falling to the ground.

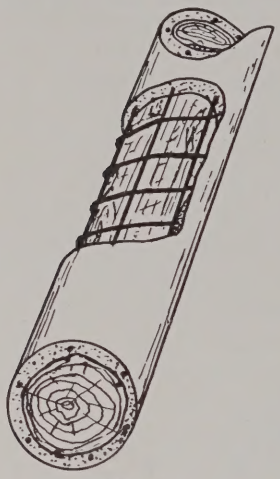


Fig. 2.

On a high-tension transmission line between Livet and Grenoble, a distance of about 20 miles, M. A. Burgeat installed the combination poles shown in Fig. 2. In the manufacture of these poles wooden poles were thoroughly dried, cleaned, and trimmed, reducing the diameter about 1 inch in every 7 feet. In a stiff cement paste covering this wooden core, $\frac{3}{16}$-inch round rods were wound in a spiral. Tied to this spiral and placed longitudinally were round rods of $\frac{1}{16}$- to $\frac{1}{8}$-inch diameter, the cross-section, number, and area of the rods depending upon the length and strength of the poles desired. The concrete covering was applied by placing the steel-encased wooden core in a form and pouring concrete around it. These poles, while strong, were cumbersome, almost as heavy as solid concrete, and required considerable time to manufacture. In addition, a wooden core is subject to organic change, and may cause cracks in the concrete, by expansion or contraction, as its moisture content varies.

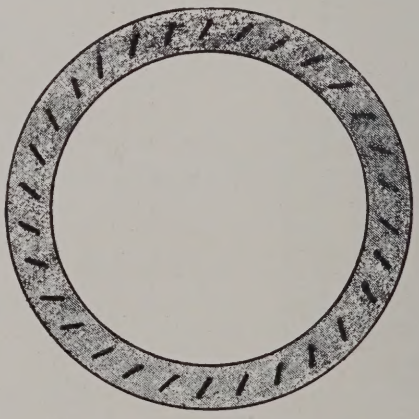


Fig. 3.

A more recent process invented by the German firm of Otto & Schlosser, at Meissen, on the Elbe, consists in manufacturing poles in revolving forms by centrifugal force. A few of these poles have been installed on the government telegraph lines in Meissen, and it is stated that thus far they have not required any maintenance expenditures. To a wet mixture of rich concrete is added finely ground asbestos fiber, and the resulting mixture is placed in a tubular form,

inside which the reinforcement of expanded metal has been fastened, and revolved at high speed. It is claimed that the centrifugal action forces the concrete to an even thickness against the reinforcement, the operation taking place in a warm room and occupying but a few minutes. By the addition of asbestos fiber the strength of the poles in tension is said to be increased.* These hollow poles, shown in cross-section (Fig. 3), have the butts filled with stones to the ground-line.

The Brescia Construction Company, of Brescia, Italy, have constructed a novel kind of pole, in lengths from 26 to 33 feet, and of ordinary telegraph, telephone, or trolley capacity. The form of construction is shown in Fig. 4; a large round bar in each of the three corners, firmly cross-tied, composes the reinforcement. The poles are cast in wooden forms and are tapered. The manufacturers of this pole claim that their product is cheaper than corresponding iron poles. These poles are unclimbable without a special attachment, which is supplied to the workmen.

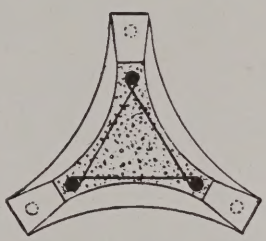


Fig. 4.

Perhaps the most remarkable process of foreign pole manufacture, known as the Swiss process, is that invented and controlled by the Messrs. Siegwart. This embodies a new idea in pole manufacture, and is a strong indication that an economical concrete pole will eventually be evolved to successfully compete, in point of first cost, with the common forms in wood and iron. The Siegwart process consists essentially of a horizontal, collapsible core of sheet-iron, with pivoted ends, carried by a movable frame which is provided with trucks. Below this revolving core is a frame supporting the continuous conveyor belt, which receives, distributes, and applies the concrete to the fabricated steel skeleton, when the latter has been fastened in the revolving core. The reinforcement consists of small rods arranged lengthwise and held accurately in place by adjustable rings, with grooves to keep the steel evenly spaced and at the proper distance from the interior and exterior concrete surfaces. On the under frame is mounted an electric motor which operates the moving parts by means of belts and worm gearing. The conveyor belt of heavy wire netting is flat, and by a system of weights is kept constantly taut, so that during a complete forward revolution of the

* The writers question whether much benefit can be derived by the addition of asbestos fiber, and in view of the experiments by L. S. Moisseiff (Am. Soc. Test. Mat., 1909) would prefer wire scrap.

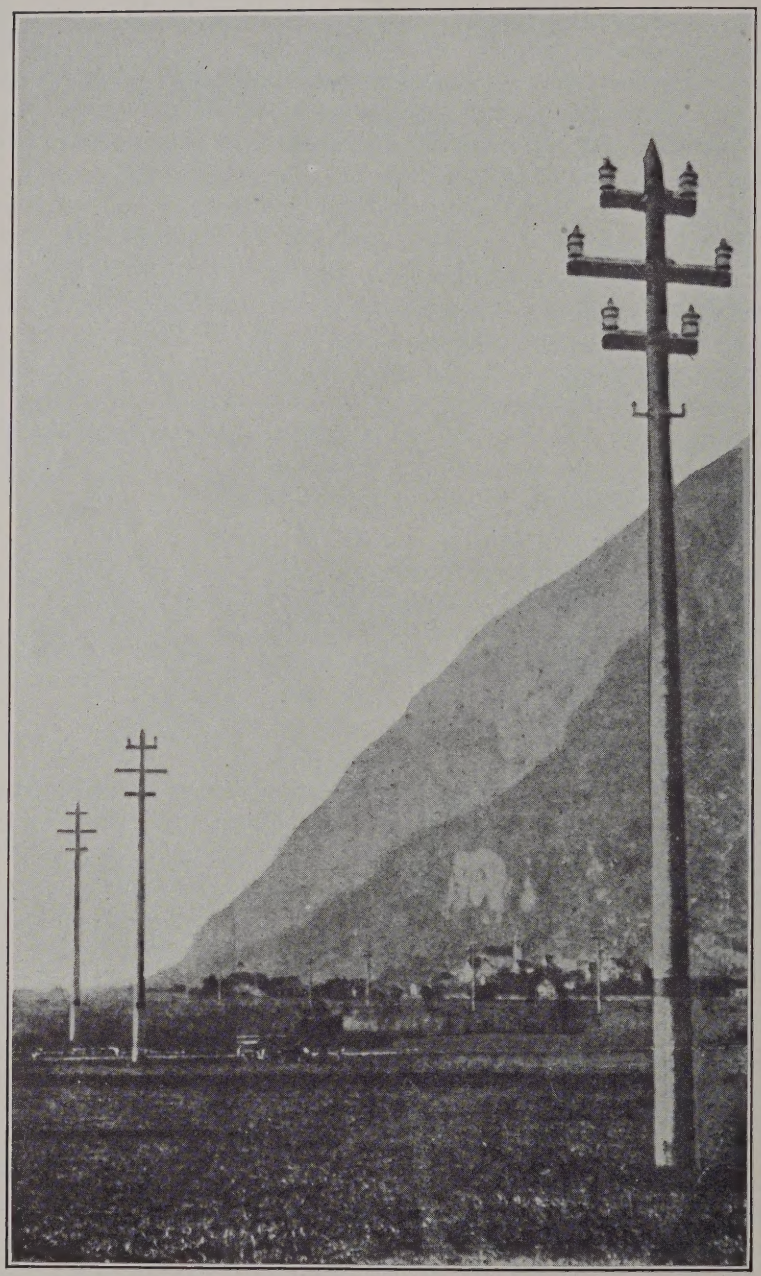


Fig. 5.—Siegwart hollow reinforced concrete poles.

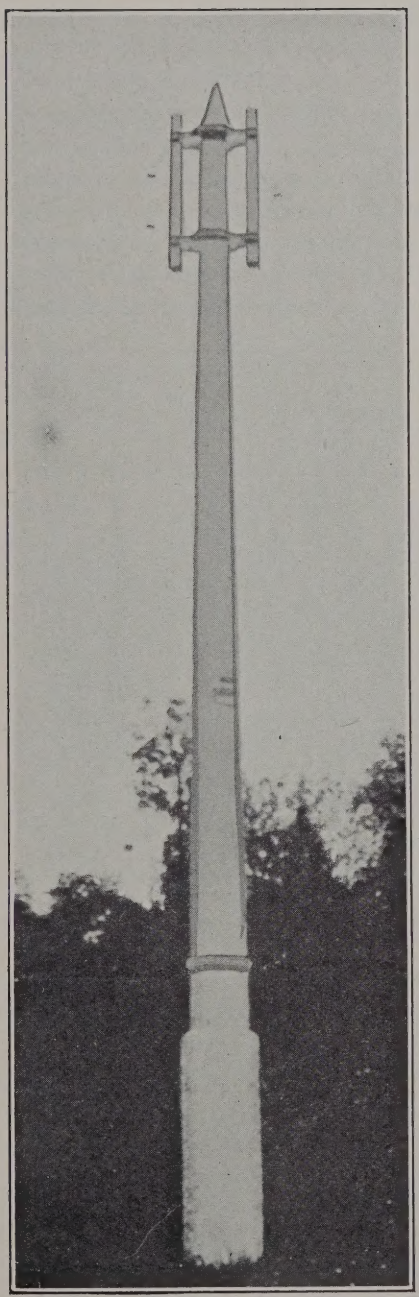


Fig. 6.

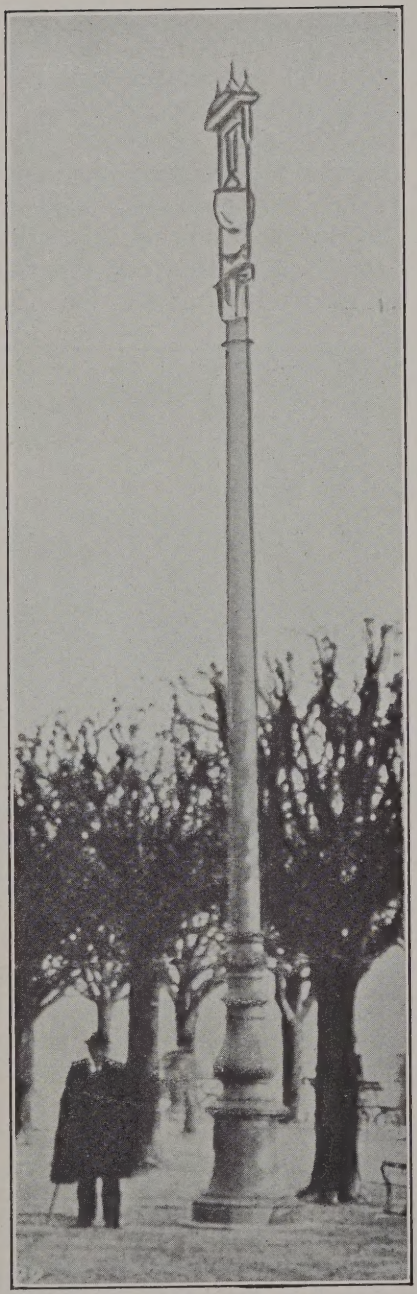


Fig. 7.

Siegwart hollow reinforced concrete poles.

core the concrete is applied or wrapped spirally around the core under pressure, one lap at a time, after which another batch is fed upon the belt and the core automatically moves forward. The under frame also carries a small mixer which supplies the concrete simultaneously with the other movements. The concrete is of a dry consistency of Portland cement, sand, and screenings, and as rapidly as applied is bound fast by canvas, wound around and smoothed out by pressing rollers which take up the slack in the canvas binding by a special contrivance. When the core has traveled the full length of the pole, it is entirely covered with concrete and canvas. The pole is allowed

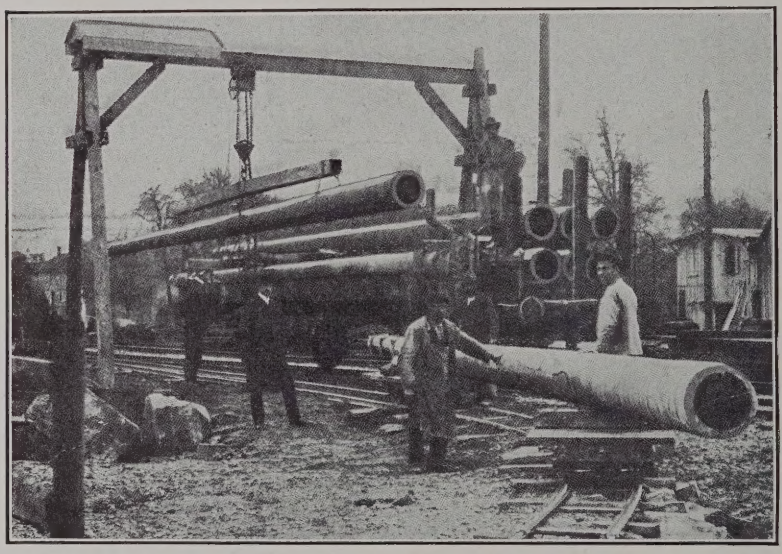


Fig. 8.—Hollow concrete poles made by the Siegwart process.

to cure in a horizontal position, from ten to fifteen hours, after which the steel core is collapsed and withdrawn. In about seven days, when the concrete has sufficiently hardened, its canvas cover is removed and the pole is ready for the cross-arms and cap. Poles of different lengths, thickness of shell, arrangement and weight of reinforcement, can be made according to the strength required. This system has produced poles up to 45 feet in length, consuming about an hour in the operation. The poles cost a little more than wooden poles, but less than iron ones, and their light weight facilitates handling and reduces freight charges. They present a good appearance, with perfectly straight lines, and are tapered or fitted

with artistic bases to conform æsthetically with their surroundings. The great advantage of these poles is that they can be made by machinery in any size and quantity.

A large number were used on the transmission line of the elevated works at Rathausen near Luzerne, the Olten-Aarburg electrical works, and the central station of the town of Zurich. These poles withstood a heavy snow-storm in Switzerland, in May, 1908, which destroyed a large number of wooden poles. The internal and external appearance of this pole is shown in Figs. 5, 6, 7, 8, and 9.

In 1903 Robert Cummings, M. Am. Soc. C. E., constructed some experimental concrete telegraph poles at Hampton, Va. These poles were about 30 feet long, with the cross-section of an equilateral triangle having 12-inch sides and reinforced with ¾-inch rods in the corners.

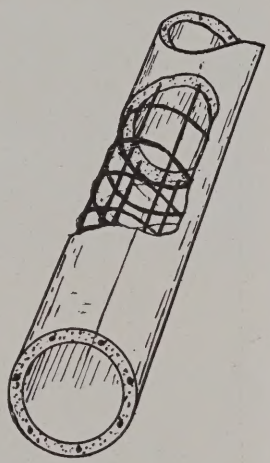


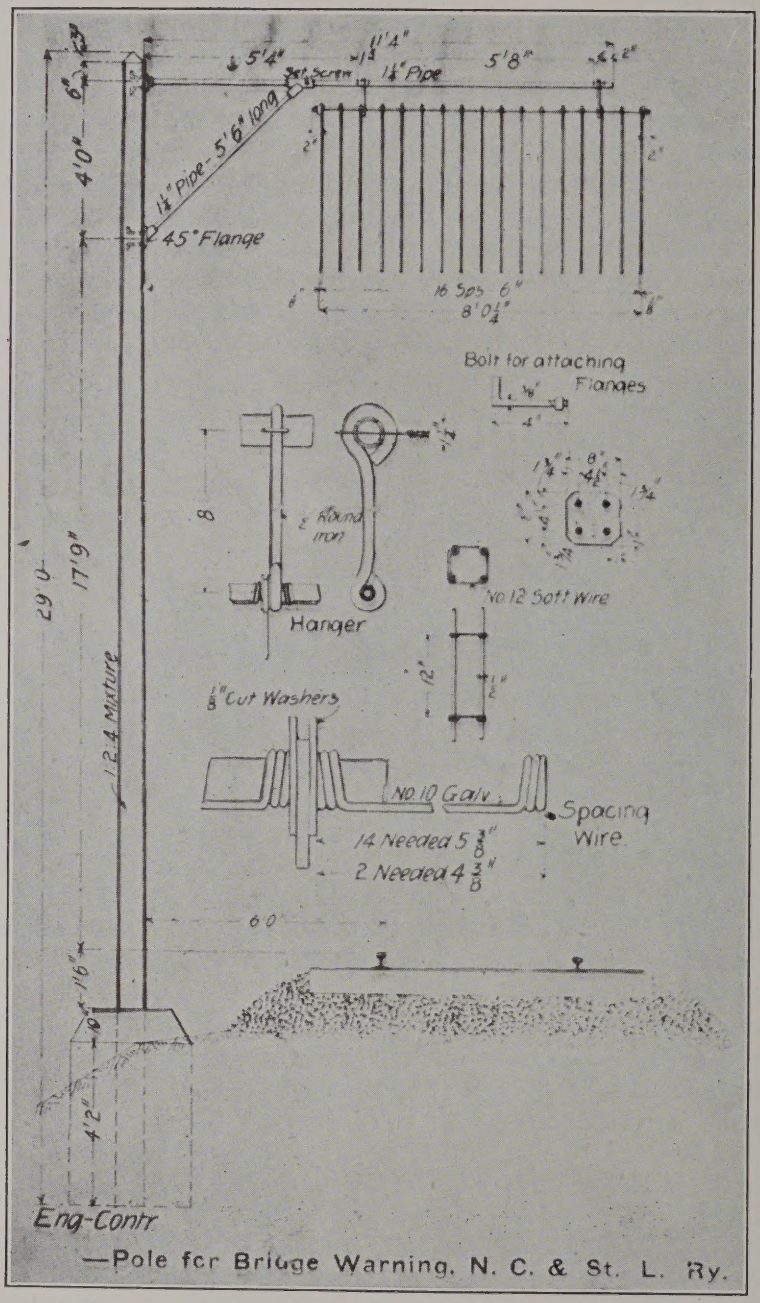
Fig. 9.—Siegwart pole.

Hunter McDonald, Chief Engineer of the Nashville, Chattanooga and St. Louis Railway, has had in use for some four and one-half years a reinforced concrete support for a standard bridge warning (Fig. 10). Some of the first supports were molded complete with pole, brace, and cross-arm, of concrete. The arm and brace were found to be too expensive, so these parts were afterward made of pipe. One of the poles, with concrete arms and braces, after four and one-half years' service, shows considerable bending, but the composite pole remains erect. For the shaft ⅓ cubic yard of platform screenings, ¼ cubic yard of sand, and 2½ bags of Portland cement were used. The base consists of 1½ cubic yards of stone, ¾ cubic yard sand, and 6 bags of cement.

Early in 1904 the United Traction Company of Albany, N. Y., began a series of experiments on reinforced concrete poles by first testing a model pole (Figs. 11 and 12).*

* Abstracted from data prepared by C. T. Middlebrook, M. Am. Soc. C. E., Albany, N. Y.

Length from wall to load..... = 6′ 0″
Cross-section at wall..... = 4″ x 4″
Cross-section at load..... = 2½″ x 2½″
Maximum longitudinal steel..... = 4.7% of section at wall
Maximum longitudinal steel (in tension) = 1.78% of section at wall
Concrete a wet mixture of 1:4 Portland cement and unscreened limestone.
Age at test..... = 6 weeks.



—Pole for Bridge Warning, N. C. & St. L. Ry.

Fig. 10.—Pole for bridge warning, Nashville, Chattanooga and St. Louis Railway.

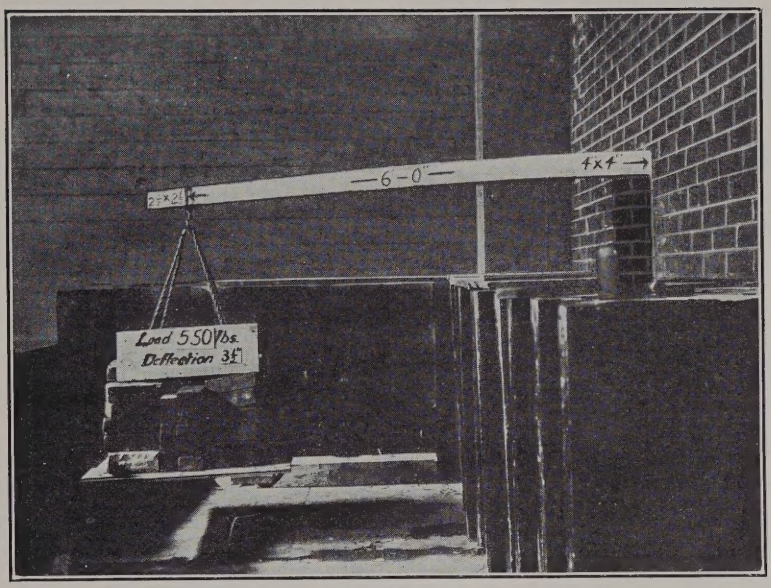


Fig. 11.—Model test pole, United Traction Co.

Fig. 12.—Model test pole, United Traction Co.

The reinforcement was composed of twelve ¼-inch square, cold twisted, steel bars, four extending the full length, four to the three-quarter point, four to the middle of the pole, and all held firmly together by a double coil of No. 12 wire with a 2-inch pitch.

TABLE 3.

Load (Lbs.).	Deflection (Inches).	Bending Moment* (Inch-lbs.).	Unit Tension in Steel* (Lbs. Per Sq. In.).	Unit Comp. in Steel* (Lbs. Per Sq. In.).	Unit Comp. in Concrete* (Lbs. Per Sq. In.).
200	¾	14,400	19,200	8,200	840
300	1½	21,600	28,800	12,300	1270
400	2 7/16	28,800	38,400	16,400	1690
550	3½	39,600	52,800	22,600	2320
600		43,200	57,600	24,600	2530†
700	4½–12	50,400	67,200	28,800	2960‡

At failure the reinforcement exerted considerable resistance to compression, after the outer coating of concrete had been crushed. The high-unit stresses are undoubtedly due to the size of the specimen, the efficient webbing, and the unit cage construction.

In consequence of the favorable showing of this model, a pole suitable for electric railway service was then made, having the following characteristics:

Length above ground..... = 28 ft.
Length below ground..... = 6 ft.
Cross-section at base..... = 13″ x 13″
Cross-section at ground..... = 12″ x 12″
Cross-section at top..... = 8″ x 8″
Maximum longitudinal steel..... = 2.5% of section
Maximum longitudinal steel (in tension)..... = 1.76% of section
Concrete, a wet mixture of 1 : 4 Portland cement and crusher run limestone, the maximum diameter of the stone being ½ inch.
Age at test..... = 6 weeks.

The reinforcement was composed of ten ½-inch and two ¾-inch square cold twisted steel bars, eight of which were arranged in a circle and enclosed in a spiral of ¼-inch twisted steel with a 6-inch pitch. Two ¾-inch and two ½-inch rods were symmetrically placed in the corners outside the circle and extended the full length of the pole; the remaining rods terminated in groups at the one-half and three-quarter points.

* Computed for comparison by the authors.
† Small cracks 3 to 6 inches long appeared on the tension side.
‡ Progressive failure by extension of steel and crushing of concrete at support. On the removal of 250 pounds the pole recovered several inches of the 12-inch deflection.

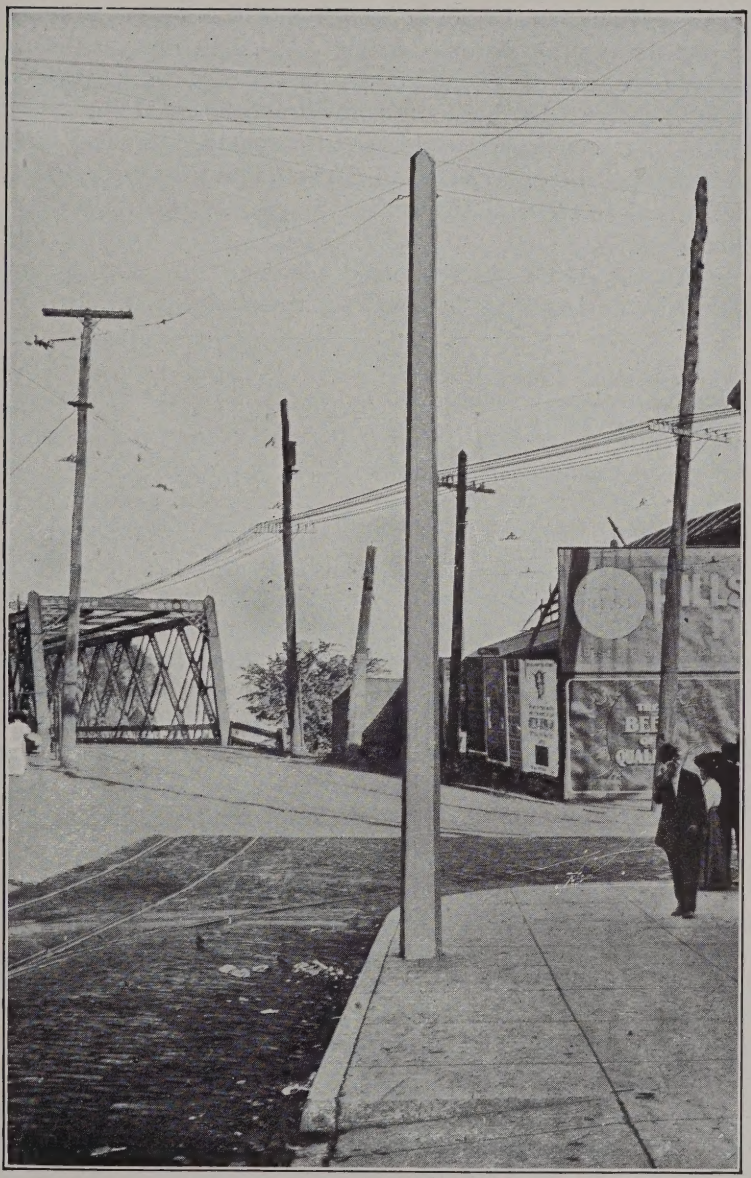


Fig. 13.—Test pole built by United Traction Co., of Albany, N. Y.

This pole was designed for a pull of 1000 pounds applied 20 feet from the ground, but, as can be seen from Fig. 13, is not apparently subjected to any considerable loading. It is now five years old and reported to be in as good condition as when erected, with no evidences of injury, except a few hair cracks, due to an excess of fine material, or to having been cast in a heated atmosphere. Its chief interest is in that it is believed to be one of the first, if not the first, reinforced concrete pole erected in the United States, for electric railway or similar use.

In August, 1904, two reinforced concrete poles were made for the Schenectady Railway Company, Schenectady, N. Y.

Length..... = 35 ft.
Cross-section at base..... = 14″ x 14″
Cross-section at ground..... = 11¼″ x 11¼″
Cross-section at top..... = 6″ x 6″
Longitudinal steel in tension..... = { 1.33% of section. (Pole No. 1.) / 1.68% of section. (Pole No. 2.)
Concrete, a wet mixture of 1 : 1½ : 3½ Portland cement, sand, and crushed limestone.
Age at test..... = 6 weeks.

The reinforcement of pole No. 1 was composed of twelve ⅜-inch square twisted steel bars, eight of which were arranged in a circle and enclosed in a spiral of ¼-inch twisted steel, with a pitch of 3 to 6 inches. Four of the bars were 28 feet long, four 20 feet long, and the four remaining bars, placed in the corners outside the circle, were full length.

Pole No. 2 was like Pole No. 1, except that the full-length bars were ½ inch instead of ⅜ inch.

An accessible point on the railway company's line was used as a casting yard. After seasoning, the poles were handled and loaded by a crane car, carried to their location and placed by the crane and an auxiliary gin pole, and are used to support a double-track span construction over a street. Wooden cross-arms, placed in gains, and supported by the usual metal brackets, are used. In handling, these poles were heavy and cumbersome, several cracks appearing, due to the large taper and the excessive deflection. The reinforcement was not adequate to withstand the strains due to lifting into position. These poles have been in place five years and appear to be in as good condition as when first installed, no signs of disintegration appearing about the cracks just mentioned.

In November, 1905, Wallace Marshall, of Lafayette, Ind., made

and tested one pole. This test pole was about 35 feet long and tapered from 10 inches square at the ground to 5 inches square at the top, with chamfered corners; the lower 5 feet, 10 inches square, was embedded in the ground. Bolts were placed at the usual heights in the forms for the attachment of cross-arms, line bracket, and telephone box. On top and in the center was placed a 1½-inch plug for an insulator pin. The mixture used was 1 part cement to 6 parts of graded aggregate, with a facing of 1½ inches of 1 : 3 mortar. The reinforcement consisted of four ¾-inch Thacher bars, 25 feet long, and four ½-inch Thacher bars 14 feet long, with a lap of about 4 feet. The forms were removed at the expiration of six days, and in thirty days the pole was planted.

For purposes of comparison a large cedar telephone pole and the concrete pole were erected 25 feet apart. A taut wire cable connected the two poles at a height of 21 feet from the ground; from the middle of this cable was suspended a barrel which received the test loads in the shape of steel rivets. As the barrel was gradually loaded with rivets the two poles began to bend toward each other. When the deflection of the poles was about 21 inches, a small check or crack appeared in the concrete pole about 10 feet from the ground, followed by others from the ground to the point of the cable attachment. At this point in the test the load was removed and its weight ascertained.

From calculations the horizontal load was found to be about 975 pounds and the stress in the reinforcement about equal to the elastic limit of the steel. When the load was entirely removed, the pole returned to its original position.

In 1906, R. E. Cummings, of Pittsburg, Pa., made for G. A. Cellar, Superintendent of Telegraph of the Pennsylvania Lines West of Pittsburg, a comparative test of two cedar and two concrete poles. The two concrete poles were hollow for two-thirds of their length, but had a solid top. The shells tapered from a thickness of 3 inches at the base to 1¾ inches at the solid portion. The poles had chamfered corners and weighed about 3500 pounds apiece, and were designed* to withstand any stress in any direction that would be produced by a line of 50 wires, each wire coated with ice to a total diameter of 1 inch; *i. e.*, a load equivalent to 1000 pounds applied one foot below the top of the pole. The two cedar poles were selected stock, and all the poles were set in a concrete foundation 3 feet

* On a 50-wire line, under a loading of ½ inch thickness of ice and a wind pressure of 8 pounds per square foot, the equivalent load would be 3100 pounds.

square and 5 feet deep. The general dimensions of the four poles and a record of the tests are given in Table 4.

TABLE 4.—LOADS AND CORRESPONDING DEFLECTION FOR FOUR POLES TESTED.

Test No.	Deflection at Top, Inches.	Load in Pounds.	Deflection at Bottom in Inches (12 inches above ground-line).	Time.	Remarks.
		30-foot Octagonal Concrete Pole, top 8 inches, base 14 inches. (Depth of concrete anchorage, 5 feet. Load applied 24′2″ from ground.)			
1 ..	3¾	1830	1/32	3:17	..
	5¾	2230	1/16	3:18	..
2 ..	½	50	1/32	..	Temporary deflection, ½ inch.
	8	2630	1/8	3:19	..
	11¾	3030	3/16	3:20	Cracks Nos. 1 and 2.
3 ..	1¼	50	1/16	..	Temporary deflection, 2 inches.
	14¼	3430	1/4	3:24	Cracks, Nos. 3 and 4.
	18	3210	3/8	3:25	Crack, No. 5, crushed at bottom.
	25½	3150	3/4	3:26	Pole broke at ground-line.
		30-foot Square Concrete Poles, top 7 inches, base 13 inches. (Anchorage and point of application of load same as before.)			
1..	¾	50	..	2:02	
	2½	1830	..	2:04	
	3½	2230	..	2:08	
2 ..	¾	50	..	..	Temporary deflection, 1 inch.
	4 9/16	2630	..	2:10	
	8½	3030	1/16	2:11	
3 ..	3½	50	..	..	
	31	3290	..	..	
	34½	3430	1/8	2:14	Crack, No. 1.
4 ..	21¼	50	..	..	Temporary deflection, 22 inches. Cracks, 2, 3, 4. Pole crushed.
	39	3690	..	2:19	Cracked at ground-line.
		30-foot Wooden Pole, No. 4, White Cedar, top 8 inches, base 14 inches. (Anchorage and point of application of load same as before.)			
1 ..	20	1830	..	11:50	
	22¼	2230	..	11:51	
	29	2630	..	11:52	
	35	2870	..	11:53	First crack.
	36½	2950	..	11:54	
	38½	3030	..	11:55	
	50	3370	..	11:56	
	56	3430	..	11:57	
	66	3494	..	12:00	Pole broke suddenly.
		Wooden Pole, No. 3, White Cedar, top 8 inches, base 14 inches.			
1 ..	14	172	..	..	
	37	2230	..	..	
	47	2530	..	11:03	Pole broke suddenly.

REVIEW

The reinforcement consisted of four ¾-inch round bars, 24 feet long, and four ⅝-inch round bars of the same length. The taper of

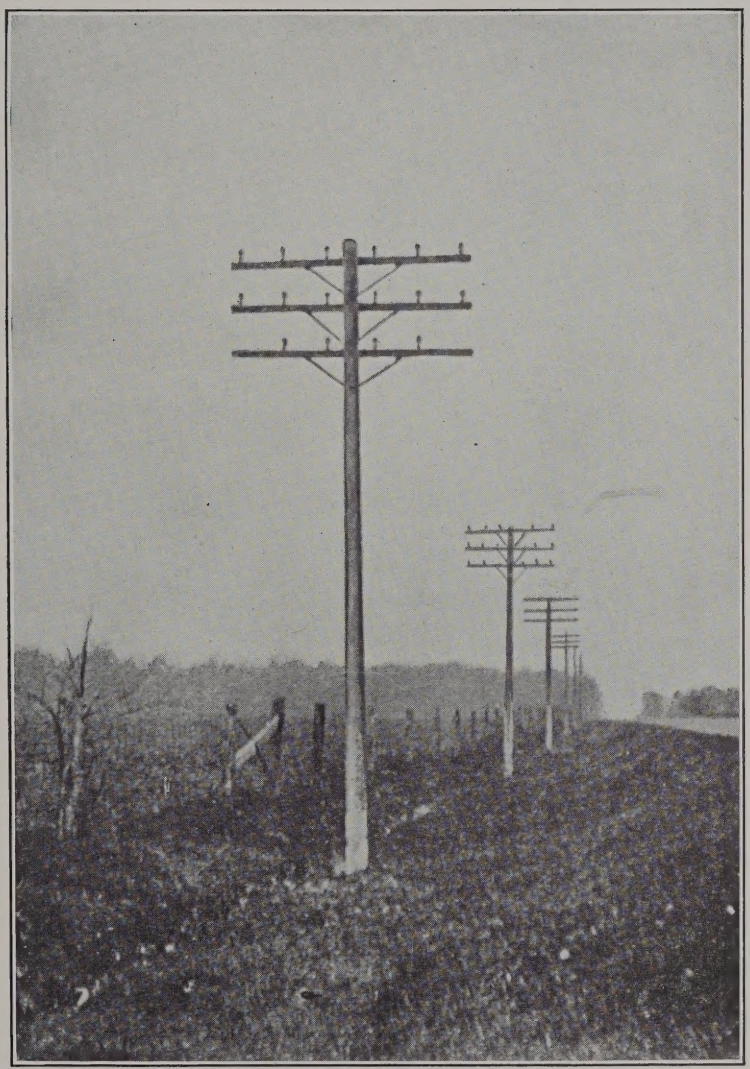
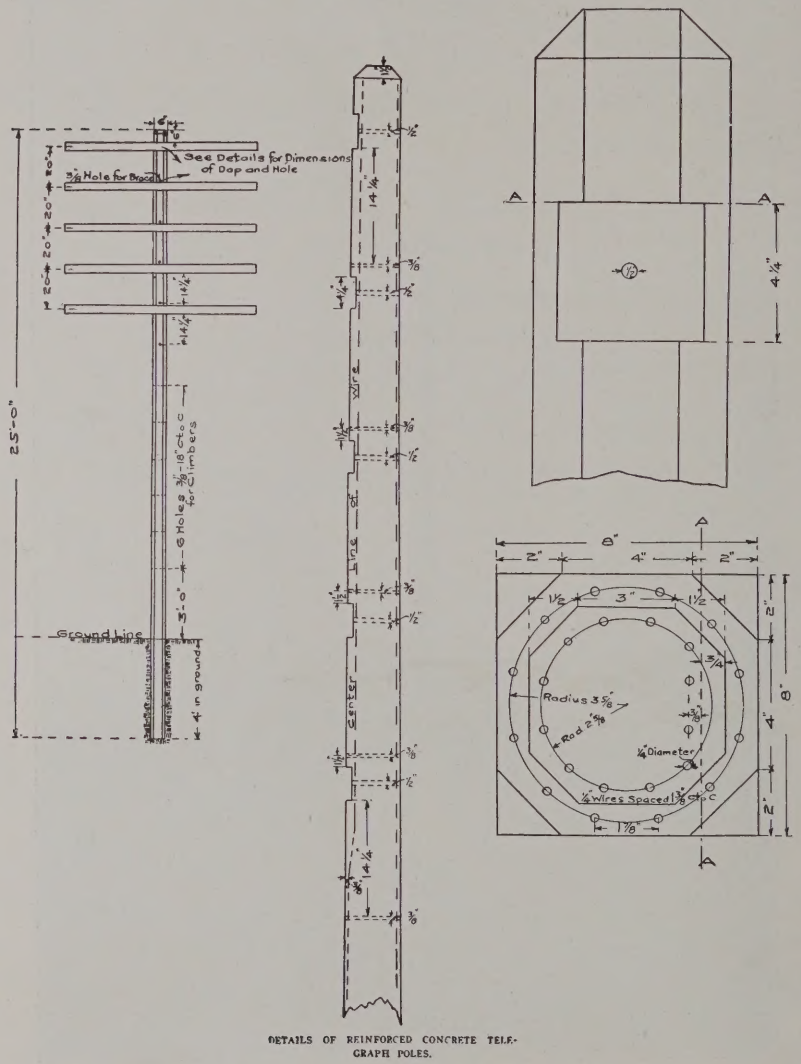


Fig. 14.—Reinforced concrete poles at Maples, Ind., P. F. W. & C. Ry.

the concrete poles was 1 inch in 5 feet. Wooden blocks into which the galvanized iron steps screwed were molded into the concrete at

proper intervals. The cross-arm braces were fastened to the pole in the same manner, by a lag-bolt, and attached to the arms by through-bolts. The load, or pull, was applied by means of a wire



DETAILS OF REINFORCED CONCRETE TELEGRAPH POLES.

Fig. 15.—Poles used at Maples, Ind., P. F. W. & C. Ry.

rope attached to an iron clevise placed around the pole 10 inches from the top and drawn over a pulley placed at the same height.

A 1 : 3 mixture was used, the poles were cast in cold weather, and suitable gravel was not obtained. A defect in pole No. 1 probably caused its early rupture. A more satisfactory result was obtained from the test of pole No. 2.

Mr. Cellar says:* "After the cement poles had been broken, the reinforcement so held them that it required almost the breaking pressure to further deflect them from their slightly inclined position. The wooden poles under strain presented the form of an arch before breaking, and when they gave way were fractured completely; but these features were lacking in the cement poles, which were very firm and did not give until they began to crush at the ground-line."

In the early part of 1906, J. B. McKim, Superintendent of the Western Division of the Pennsylvania Lines West of Pittsburg, built a line of 53 reinforced concrete telegraph poles near Maples, Indiana, along the Pittsburg, Fort Wayne and Chicago Railway. These poles vary in height from 20 to 28 feet, the length of pole above ground varying with the profile, so that the telegraph line is parallel to and at a constant distance above the track. The poles are quite small in cross-section and are of minimum weight. They have now been in use four years and show no signs of deterioration. (Figs. 14 and 15.)

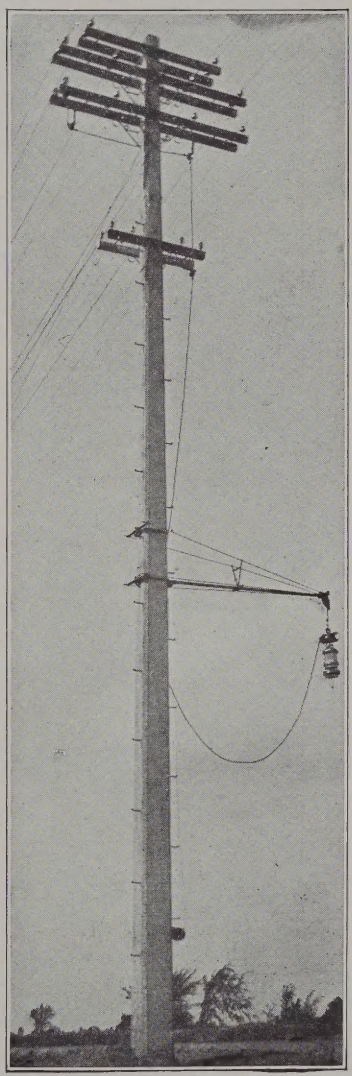


Fig. 16.

At the present time several hundred concrete poles are used by the various transmission companies, in distributing to interior points the current generated by Canadian

* "Experiments with Concrete Telegraph Poles," G. A. Cellar, Proc. Rwy. Tel. Supts., 1907.

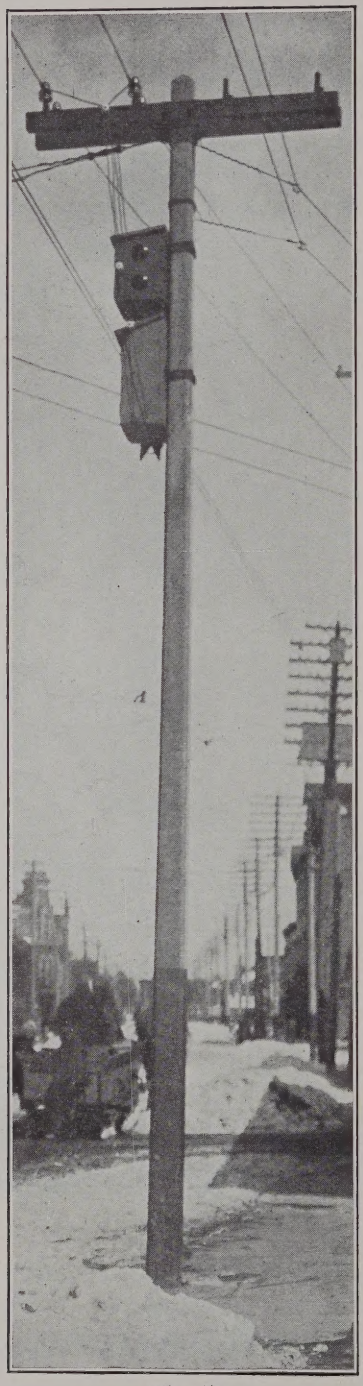


Fig. 17.

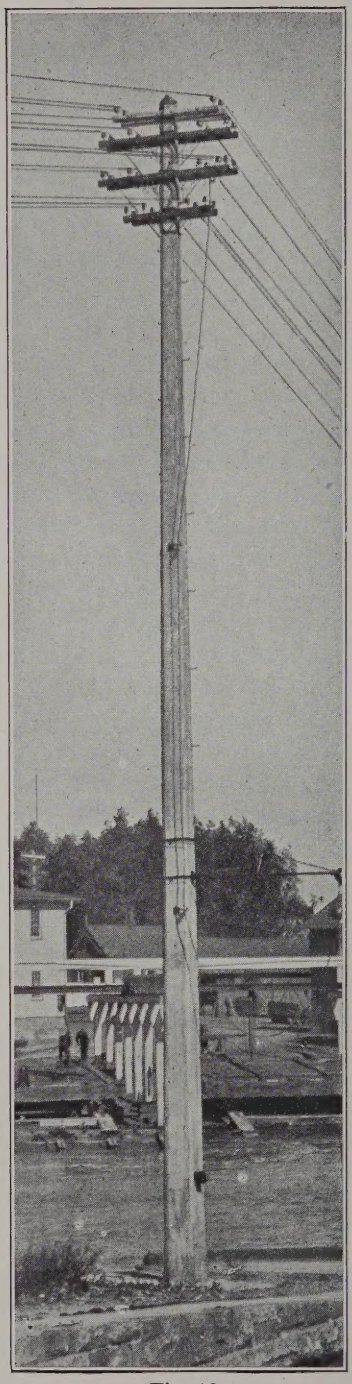


Fig. 18.

water-powers. Examples of the poles erected about Niagara Falls and the Welland Canal are shown in Figs. 16, 17, 18, and 19.

In 1903 the Concrete Pole Company, of St. Catherines, Ontario, built about twenty poles for the Niagara Falls Power Company, on their main line to Buffalo. A little later a number of transmission poles were built for the Canadian Niagara Power Company, at Chippewa, and for the Ontario Power Company, at Port Robinson and Welland.

In 1906 the same company constructed a power line, for a distance of 12 miles, for the Hamilton Power, Light and Traction Company. The poles are 35, 40, 45, and 60 feet in total length, the longer poles being used at road and other crossings. They are spaced about 200 feet apart and carry 00 B. & S. gauge copper wires, forming two 3-phase circuits of 40,000 volts. These poles sustained safely a test pull of 2000 pounds applied at the top of the pole. As a part of this work two towers 150 feet in height —believed to be the highest concrete monoliths in existence—were successfully constructed on each side of the old Welland Canal, to carry a transmission line to St. Catherines, Ontario. These towers are guyed, but without guys can safely withstand a pull at the top of 2000 pounds. They are embedded 8 feet in a heavy concrete base, measure 11 inches square at the top and 31 inches square at the bottom, and carry sixteen No. 1 bare copper wires on glass insulators. The cross-arms are of concrete, 3¼ inches by 4 inches by 10 feet long. A platform 10 feet long by 5 feet wide, at a convenient distance beneath, enables workmen to make inspection and adjustments with safety. The canal span is only 76 feet, but the approach span is about 300 feet. One of the towers, in addition to carrying a heavy weight of wires arranged

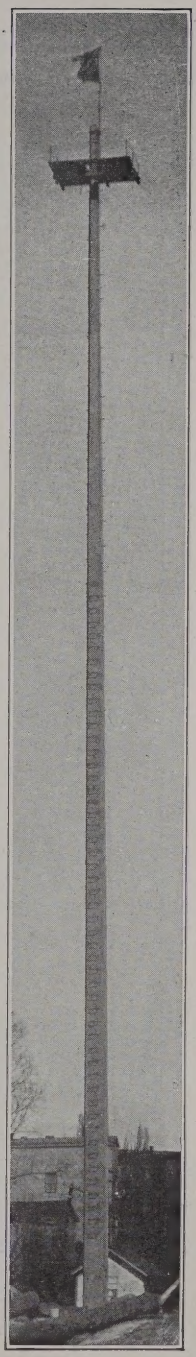


Fig. 19.—150 ft. pole—31 in. base, 11 in. top; weight 45 tons. Welland Canal crossing poles, H. P., L. & T. Co.

vertically on two frames, is at a right-angled bend and sustains a heavy angular pull.

In 1906 A. C. Chenoweth, of Brooklyn, N. Y., constructed

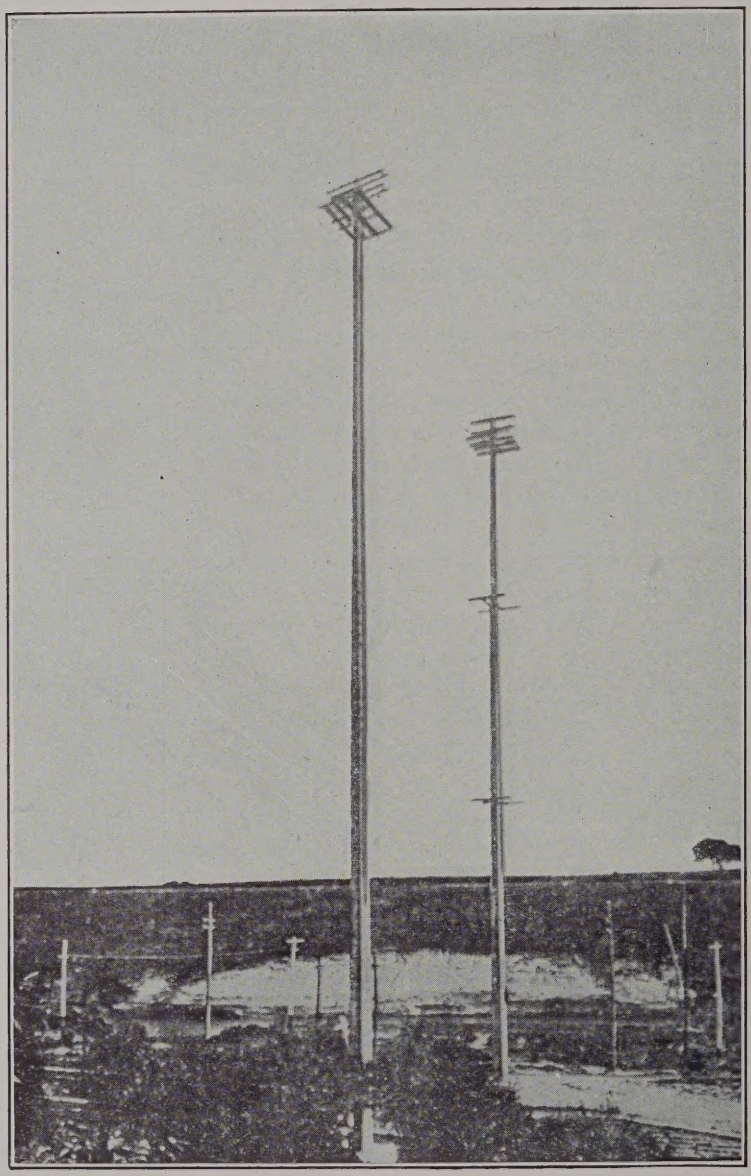


Fig. 20.—Welland Canal crossing, H. P., L. & T. Co.

some concrete poles 60 feet long with a base 14 inches in diameter, designed to carry a direct pull of 16,000 pounds and the torsional effect of an arm 4 feet long carrying 8000 pounds. These poles carried a 500-foot span of 4-inch direct-current transmission cable, and cost about $2.50 per lineal foot.

The Chenoweth concrete pole is rolled by a specially designed machine, and may be made hollow and with a taper. It is formed by rolling steel wire mesh and longitudinal rods, covered with concrete, into a coil.

Reinforced concrete towers of rather huge proportions were erected in 1906 for the West Penn Railway Company for their transmission line crossing over the Monongahela River at Brownsville, Pa. One structure is 150 feet in height, supporting a cable span of 1014 feet, at an average height of 105 feet from the base of the tower. The tower itself is guyed to an anchor tower in the rear. The larger main tower has a foundation 30 feet square and is 8 feet 6 inches square at the top of the foundation. These immense poles are square, hollow in cross-section, and tapering. Small I beams constitute the reinforcement of the tower, while the base is composed of a large slab, reinforced by a meshwork of rods.

In 1907 a number of the Seigwart poles were tested at the Olten Aarburg electric works. One of the poles tested carried eight No. 8 wires, the pole being located at a bend in the line. Of a total length of 38 feet, 4 feet 8 inches were embedded in the foundations. In the test the pole was placed horizontally between two large concrete blocks, a pulley block being attached 30 feet from the point of grip, corresponding to the top of the foundation or surface of the ground. A dynamometer was used to measure the load. In Table 5 are given the results of the first test, but it should be noted that during this test, that portion of the pole held within the foundation blocks twisted, and when the strain was removed the pole returned to within an inch of its original position, so that the total deflection was in reality more nearly 2.52 inches.

TABLE 5.

Pull in Pounds.	Deformation in Inches.	Permanent Deformation.
88	0.00	....
220	0.12	....
440	0.32	....
660	0.60	....
880	1.12	....
1100	1.68	....
1320	2.08	....
1540	3.52	....

A second test was made on the pole by applying the load in two increments, the first, of 88 pounds, resulting as before in zero deflection, and the second load, of 1540 pounds, giving a deflection of 2.8 inches. When the load was removed, the pole returned to its first position.

As a final test the pole was loaded as shown in Table 6.

TABLE 6.

Pull in Pounds.	Deformation in Inches.	Permanent Deformation.
1540	2.84	....
1760	3.6	....
1980	5.4	....
2200	6.2	....

Under the extreme condition of loading it was found that the foundation had yielded, so that the actual final deflection amounted to about 4.8 inches.

In March, 1907, the United Traction Company, of Albany, made further tests of several reinforced concrete poles.* These poles were all alike in cross-section and length, the arrangement of cross-arms and kind of reinforcement distinguishing the different poles.

Pole No. 1, illustrated in Figs. 21, 22, and 23, is a type of concrete pole between trolley tracks which, in addition to carrying on the lower arm a catenary suspension trolley line, supports two feeder wires and a 3-phase transmission line. Standard insulator pins were used, the one at the top cast in place, and the others placed in cored holes in the arms. This construction is entirely of reinforced concrete. The reinforcement of the pole proper was of the rectangular cage construction. Four $\frac{3}{4}$-inch square bars, running the full length of the pole, were fabricated into a square, by right and left turns of a double coil of No. 12 wire, having a pitch of about 3 inches, and tied frequently to the main rods. The lower arm was reinforced with four $\frac{1}{2}$-inch square twisted bars and No. 14 wire, having a right and left pitch of 2 inches. In the upper arms and brackets four $\frac{3}{8}$-inch bars and No. 14 wire were used. The steel of the cross-arms and brackets was tied to the main pole reinforcement, thus constituting a unit skeleton frame. The minimum distance between the main bars and the surface of the concrete was 1 inch.

After the pole had attained an age of fifty-five days, a horizontal load or pull was applied about 20 feet 6 inches from the ground, or 6 inches above the lower arm.

* Abstracted from data prepared by C. T. Middlebrook, M. Am. Soc. C. E.

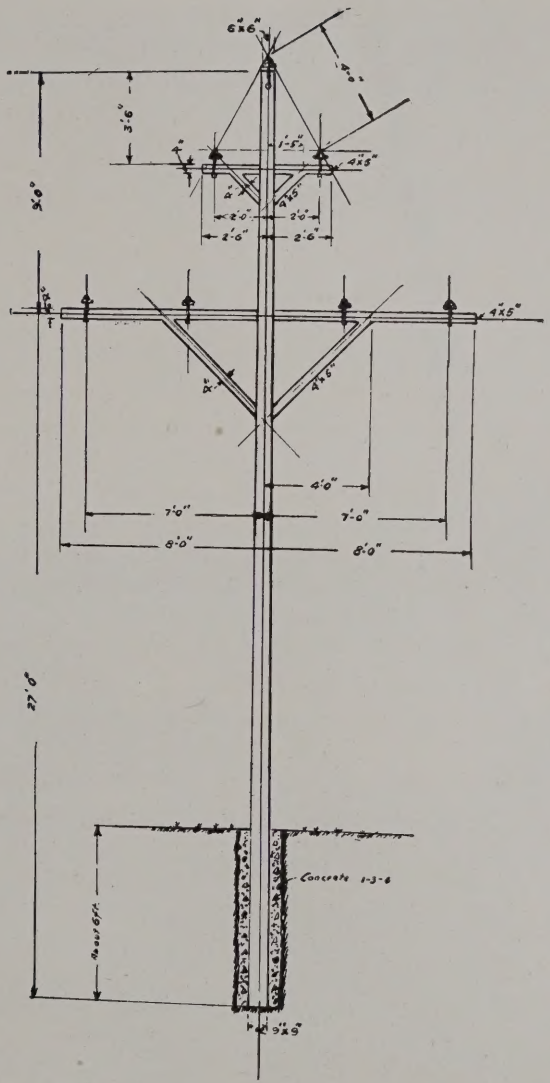


Fig. 21.—Outline plan of reinforced concrete pole, designed for electric railway service, double track carrying high-tension transmission line. Pole designed to take pull of 500 lbs. at 21 feet from ground with factor of safety of four, or 1000 lbs. at two-thirds the elastic limit of the steel; steel, 400 lbs.; concrete, 20 cu. ft. Concrete proportions: 1 part Portland cement, 1½ parts sand, and 3 parts crushed stone, not over one-half inch in size. Weight of pole, 3200 lbs.

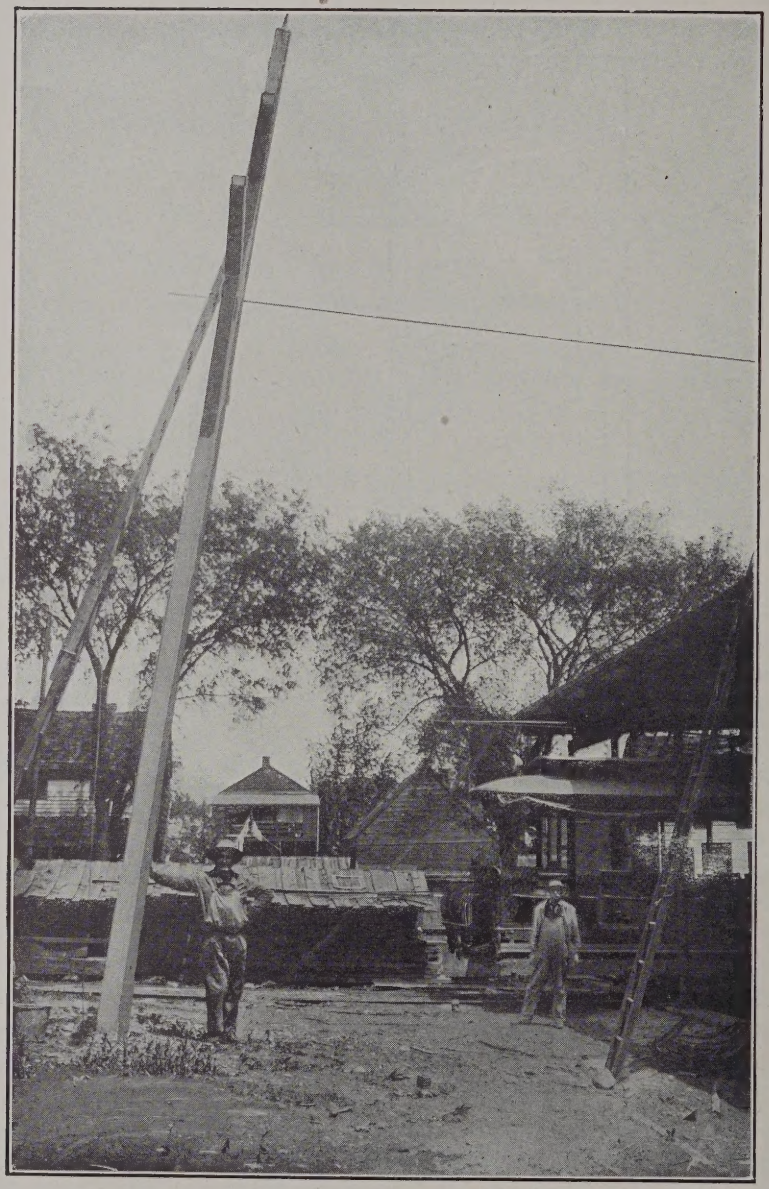


Fig. 22.—Test of United Traction Co. pole.

REVIEW

At a convenient point a dynamometer was placed to measure the various loads corresponding to the deflections in the pole. A plumb-bob was suspended from a point on the pole at the same distance above the ground as the point of application of the load.

TABLE 7.

Load (Pounds).	Deflection (Inches).	Bending Moment (Inch-Pounds).*	Unit Tension, Steel.*	Unit Compression, Steel.*	Unit Comp., Concrete.*	Arm.
200.....	$1\frac{1}{8}$	49,200	7,048	2,420	320	20.5 ft.
425.....	$2\frac{7}{8}$	104,550	15,000	5,100	640	....
600.....	$5\frac{5}{8}$	147,600	21,100	7,200	900	....
800.....	8	196,800	28,200	9,600	1,200	....
1000.....	$10\frac{1}{2}$	246,000	35,200	12,100	1,500	....

The compression of the soil at the foot of the pole increased the total observed deflection. An opening $\frac{1}{2}$ inch wide appeared in the ground at the base of the pole on the tension side. When all the load was removed, the pole returned to within $1\frac{1}{4}$ inches of its first position, this amount undoubtedly being the measure of the compression of the soil. When the load reached 1000 pounds, a few minute cracks appeared following the hair cracks. These cracks were cut into with a cold chisel, before the load was removed, and water was applied in an attempt to ascertain to what extent it would be absorbed by the cracks. The cracks did not appear to be more than $\frac{1}{4}$ inch deep.

After three weeks had elapsed this pole was again subjected to a second test, the load being gradually increased to 1000 pounds, with about the same deflections as in the first test. Another increment was added, making the load 1375 pounds, and producing a deflection of 13 inches. No cracks were observed except those noted in the first test. While under a load of 1375 pounds the cracks were again examined with a cold chisel for a depth of $\frac{3}{8}$ inch. Beyond this depth there were no evidences of cracks. Mr. C. T. Middlebrook, who conducted the tests, remarks that "as the stretch of the steel on the tension side under this pull amounted to about 0.00167 of its length, or about $\frac{1}{50}$ of an inch per foot, it would appear that the particles of aggregate must adjust themselves, under tensile stress, in such a manner as to render the detection of cracks in the body of the concrete, even when comparatively near the surface, much more different than in the rich mortar

*Computed for comparison by the authors.

surface itself." On the removal of the load, the pole returned to its original position, except for the permanent deflection due to the

Fig. 23.—United Traction Co. pole.

compression of the soil. These tests were considered successful for a pole under horizontal loading.

REVIEW

The cross-arms were tested by loading each arm at a point 7 feet 6 inches from the center of the pole, with increasing weights up to 800 pounds, without any serious torsional effects. It was concluded that the spiral webbing was ample provision against strains of this nature.

Pole No. 2 was like pole No. 1, except for the arrangement and quantity of steel. Four ⅞-inch round Bessemer rods were used with right and left binding coils; but in order to test the effectiveness of the tie between the main bars and the coils, full-length ¾-inch [’s were clamped to the corner bars by shrinking on ½-inch by ⅛-inch steel straps at 1-foot intervals. The combined area of two bars and two channels was 1.52 square inches, effective in tension, as compared with 1.12 square inches of pole No. 1.

It is quite probable that the total area of 1.52 inches was not entirely effective on account of the difficulty in getting the concrete mixture between the ⅞-inch bar and its enclosing channel, and thus assuring a proper bond.

Compared with pole No. 1 the deflections were much greater for the same loads, and at 1000 pounds there were signs of incipient failure, a crack opening on the tension side 1 foot from the ground-line. When 1300 pounds was reached, the largest reading for this pole, the crack widened, apparently indicating the yield point of the steel. On removing the load the pole did not recover more than a few inches of its deflection, remaining 30 inches out of plumb.

TABLE 8.—POLE No. 2.

Arm. (feet).	Pull (Pounds).	Bending Moment.	Deflection (Inches).	Unit Tension, Steel.	Unit Comp., Steel.	Unit Comp., Concrete.
20.5	1,000	246,000	13	27,000	8,900	1,240
20.5	1,300	319,800	15	35,000	11,500	1,610

There were no evidences of failure of the concrete on the compression side opposite the point of failure of the steel, although at the latter point the concrete was badly disintegrated, the crack extending half-way through the pole. The spiral winding hoops probably increased the compressive resistance of the concrete.

The poles were made in substantial 2-inch spruce forms, well dressed and oiled and supported horizontally. They were cast and cured in a dry, steam-heated atmosphere, the mixture being 1 : 4 Portland cement and limestone screenings, ½-inch maximum, with a

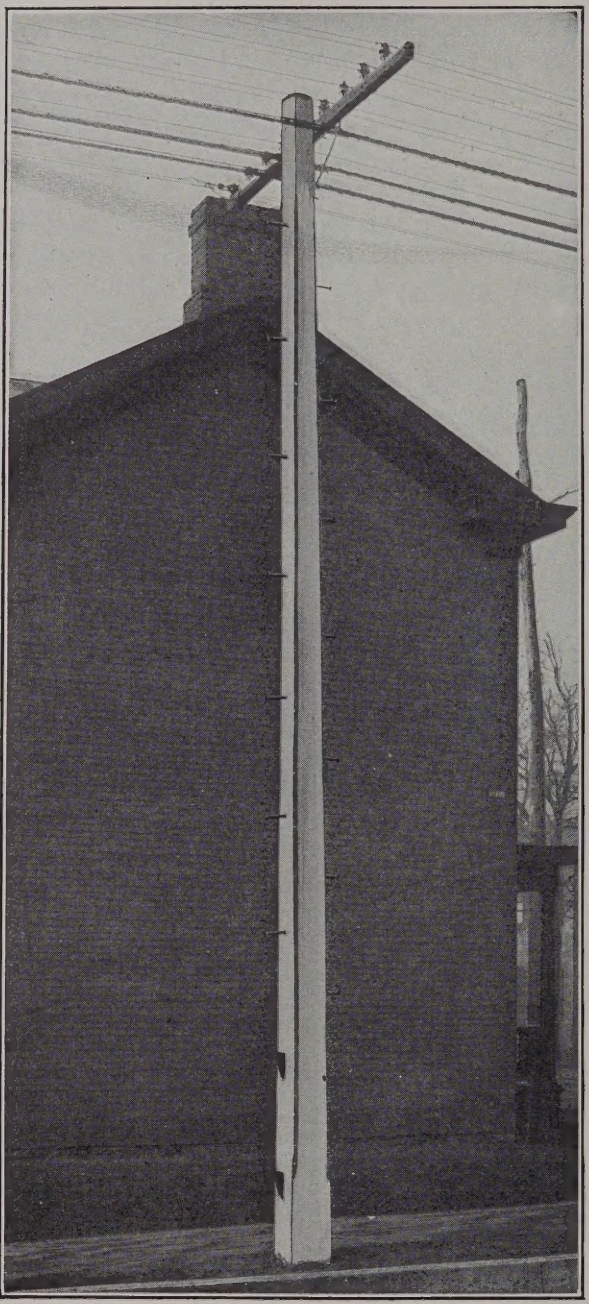


Fig. 24.—Poles at New Brighton, Pa. P. R. R.

small amount of sand. All the poles showed hair cracks, probably due to the atmosphere in which they were cured and to the presence of loam or other impurities in the crusher dust. In setting, the poles

Fig. 25.—Oklahoma Gas and Electric Co. line.

were embedded 6 feet in the ground, and some concrete was placed around the pole to increase its resistance.

From a comparison of the tests of poles No. 1 and No. 2, it would appear that the cold twisted square bars in pole No. 1 had about twice the elastic limit of the commercial Bessemer rounds in pole No. 2,

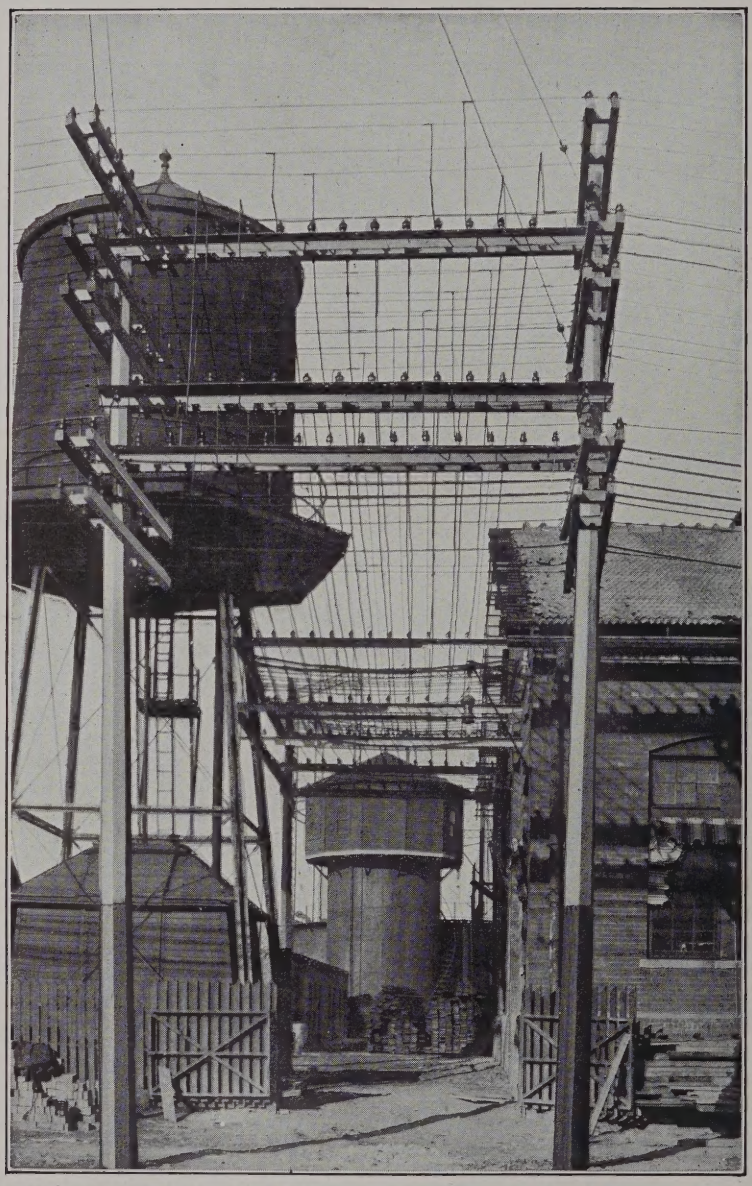


Fig. 26.—Oklahoma Gas and Electric Co. poles.

and gave better results, though the percentage of reinforcement was 0.23 per cent. less.

TABLE 9.—POLE No. 1.

Normal Pull (Pounds).	Arm (Feet).	Unit Tension (Pounds).	Unit Comp. (Pounds).	Unit Concrete Comp. (Pounds).
550	20.5	20,000	8,000	1,000
1375	20.5	50,400	19,800	2,600

The elastic limit of the cold twisted steel bars was about 55,000 pounds per square inch. Inasmuch as the surface cracks developed

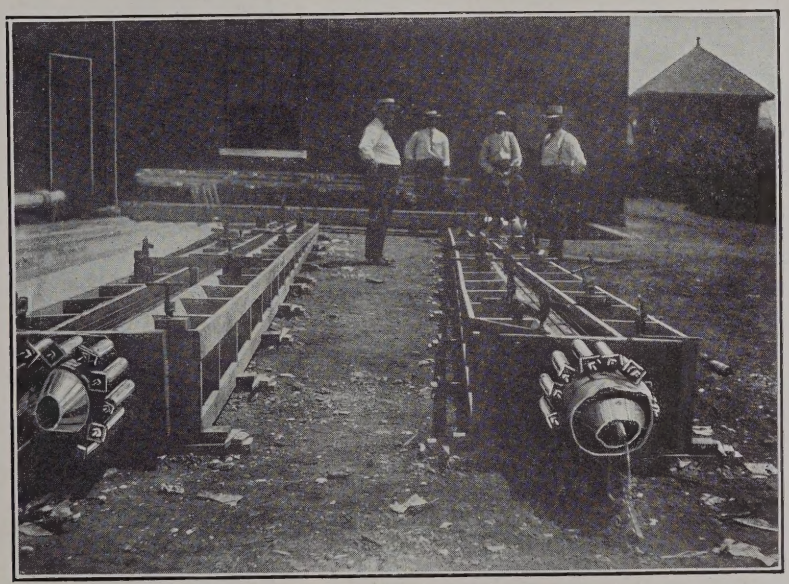


Fig. 27.—Oklahoma Gas and Electric Co. forms.

at 1375 pounds were not large or deep enough to admit moisture, it would seem economical to use a bar having a mechanical bond and steel of a high elastic limit. Pole No. 1 was designed for a normal pull of 550 pounds.

In the summer and fall of 1908 G. A. Cellar, superintendent of telegraph of the Pennsylvania Lines West of Pittsburg, had designed and erected a reinforced concrete pole line through the town of New Brighton, Pa. The poles are square in section with chamfered corners. They are 35 feet long, with a width at the top of 6 inches and

a width at the base of 14 inches; the slope increasing toward the butt 1 inch for each 5 or 6 feet of length, depending upon the conditions of loading used and the allowable stresses.

The Oklahoma Gas and Electric Company have installed as a part of their permanent construction a number of reinforced concrete poles, shown in Figs. 25, 26, 27.

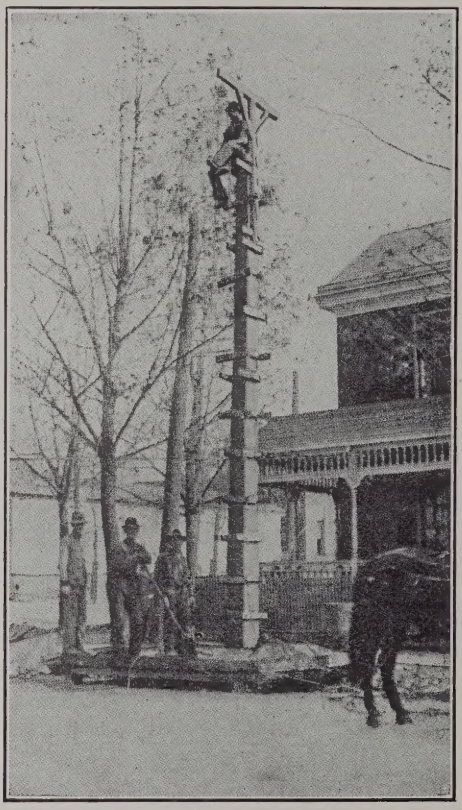


Fig. 28.—Filling form with concrete.

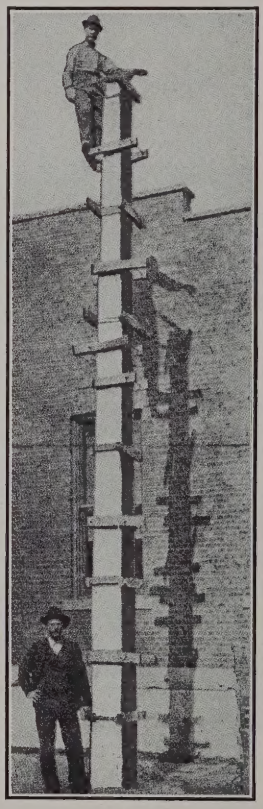


Fig. 29.—Form after being filled with concrete.

The poles are hollow, hexagonal in section, measuring for a 35-foot pole 7 inches maximum diameter at the top and 16 inches at the bottom. The reinforcement consists of twelve ¼-inch high-carbon steel rods, with mechanical bond, arranged symmetrically about the center. At each end of the pole the rods are held in position by passing the ends through top and bottom plates and bending down the

ends. Another plate is placed on the turned-down ends and bolted to the spacing plate. Before the butt plate attachment is made the rods are stretched and anchored by 10-inch pieces of rod with an eye made at one end and threaded at the other, the reinforcing rods being hooked into the eyes. Short pieces of pipe over the stretchers

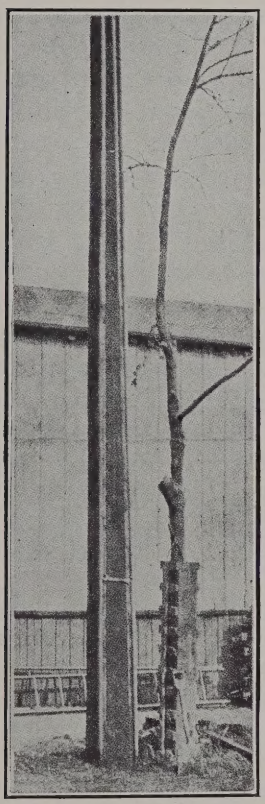


Fig. 30.—Face of form removed after four days.

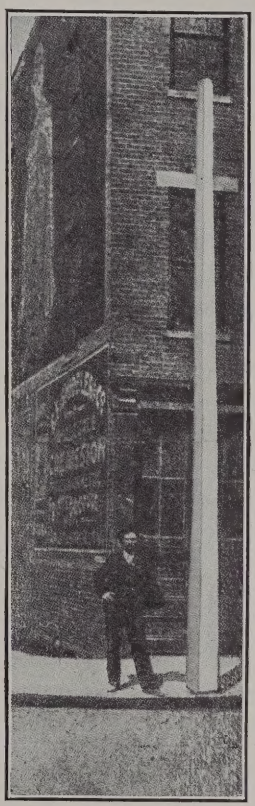


Fig. 31.—Thirty-foot concrete pole.

butt against the steel plate. Nuts and washers are screwed down on the outer end of the pipe, producing an initial tension in the rods. The core is suspended by wires in the center of the outer forms and is covered with one thickness of building paper; concrete of a 1 : 2 : 3 mixture is then added. Three parts of chats or zinc tailings are

mixed with the cement and sand and are obtained at reasonable cost and quantity from the local zinc mines of southwestern Missouri. This company claims to make the 35-foot poles at a cost of $10 with cement at $1.50 per barrel, sand at $2.00 per cubic yard, chats at $2.00 per cubic yard, and labor at $2.00 per day.

The American Concrete Pole Company of Richmond, Indiana, have constructed for the local traction company, the Terre Haute, Indianapolis and Eastern Traction Company, some forty-three reinforced concrete poles, varying from 14 to 60 feet in height. Figs. 28, 29, 30, and 31 show the various stages of construction. For poles under 35 feet in height this company claims that it is economical to mold the poles on the ground and erect by derrick. Poles exceeding 35 feet in height are cast in their final vertical positions, so that when the forms are removed the pole is ready for service. The forms are constructed of wood and iron so put together as to prevent warping and give a smooth exterior surface. One side of the form is removable to aid in placing firmly and accurately the four longitudinal reinforcing rods. A continuous spiral of binding wire extending from top to bottom forms the web reinforcement.

In this locality it is claimed that a 45-foot pole ready for use costs $25, while in the same locality a dressed cedar pole in position costs $22.50. Under local conditions this company makes the following comparative estimate of the cost of work actually done in the construction of trolley poles.

TABLE 10.—COMPARATIVE ESTIMATED COST OF REINFORCED CONCRETE AND CEDAR POLES. (Cost of Concrete Poles is without Royalty.)

CONCRETE POLES.

Length.	Top.	Bottom.	Steel.	Cu. Ft. Concrete.	Cost Steel.	Cost Concrete.	Cost Bail Wire.	Labor.	Total Cost.
Feet.	Inches.	Inches.	Inches.						
25	6	10	3/8	16	$1.57	$2.24	$1.20	$4.70	$9.71
30	6	11	1/2	21	2.29	2.94	1.20	5.20	11.63
35	6	12	5/8	26	3.91	3.64	1.20	5.70	14.45
40	7	15	3/4	36	6.31	5.04	1.50	7.20	20.05
45	7	16	7/8	43	8.56	6.02	1.50	8.70	24.78
50	7	17	7/8	50	9.50	7.00	1.80	10.20	29.50
55	7	18	1	56	13.34	7.84	1.80	11.95	34.95
60	7	19	1	61	14.56	8.54	1.80	14.70	40.60

CEDAR POLES.

Length.	Top.	F. O. B.		Labor.	Total Cost
Feet.	Inches.				
25	7	$2.60	(Dressed, graved, ruffed, bored,	$1.50	$4.10
30	7	6.25	hauled, and set.)	2.00	8.25
35	7	8.75		2.40	11.15
40	8	12.00		3.50	15.00
45	8	17.20		5.00	22.20
50	8	20.20		6.50	26.70
55	8	24.80		8.50	33.30
60	8	29.75		10.00	39.75

The American Concrete Pole Company, of Richmond, Ind., made a comparative test of one of their 30-foot poles and a cedar pole of the same size. The pole was of square cross-section, 7 inches at the top and 12 inches at the ground-line. The base of the pole was embedded in the ground for a distance of 5 feet and thoroughly braced. The reinforcement consisted of four $\frac{5}{8}$-inch twisted steel rods of high elastic limit, bound together with No. 9 binding wire.

TABLE 11.—CONCRETE POLE.

Pull in Pounds.	Deformation in Inches.	Permanent Deformation.
840	6	....
1780	17	....
2800	30	....
3640	36	....
7200	60	....

Pole deflected over 6 feet before failing.

TABLE 12.—CEDAR POLE.

Pull in Pounds.	Deformation in Inches.	Permanent Deformation.
840	11	....
1780	33	....
2200	42	....

Pole broke at last load at ground-line.

In the final stages of the test, the concrete crumbled, allowing the rods to bend.

Design of Concrete Poles.

Before entering upon any detailed discussion of design, it is necessary to consider briefly the forces acting upon a pole line and the character of service required of its component parts. As already stated, the function of the pole is that of a cantilever beam, rather than a column. The external forces are due to dead, ice,

and wind loads, which with the exception of the pressure on the pole, must be transmitted to the pole by the wires.

The weight of the wires and their coating of sleet, together with the weight of cross-arms, insulators, and the pole itself, is a vertical load, which the pole carries as a column. The pressure of the wind, on the wires whose diameter is increased by the sleet, and upon the pole structure, is assumed as acting horizontally and at a right angle with the line. The above vertical and horizontal forces act together upon the pole, but since the horizontal forces are applied at the wires, and, therefore, near the top of the pole, their effect is much greater than the effect of the vertical forces.

In the case of a pole placed at a bend in the line, there must be added to the foregoing the horizontal component of the tension in the wires, *i. e.*, the maximum tension multiplied by twice the sine of one-half the angle of the bend.

Again, in case the sags in adjoining spans are not so adjusted as to balance the tension of the wires either side of the pole, there will be an unbalanced pull in the direction of the line, which must be considered in conjunction with the vertical and horizontal forces first mentioned. Unbalanced tension may also be produced by unequal ice and wind loads in adjoining spans.

If it is further assumed that all, or part, of the wires may be broken by excessive loading, faulty material, or by burning, then the pole must withstand a longitudinal force equal to the tension in the wires in the unbroken span. This condition is fortunately very unusual, and is not generally taken into account on intermediate poles.

The usual attachments for fastening line wires to the insulators do not have sufficient strength to develop the ultimate stress of the wire, and, therefore, a broken wire would pull through into the adjoining spans before exerting its maximum tension upon the poles. As a matter of economy, it is usually better to dead-end the wires and poles at intervals and confine the effects of broken wires to the section in which the break occurs, rather than make every pole and attachment of sufficient strength to dead-end the line.

In addition, it can be shown by a rather complicated mathematical demonstration that, owing to certain properties of the catenary curve, a slight bending in a number of poles will balance the tensions in adjoining spans.

"Omitting from consideration the effects of tornadoes and cyclones, it is necessary to determine, or assume, the maximum velocity of the wind, for general practice, or for any particular locality. . . .

REVIEW

The records of the United States Weather Bureau—omitting tornadoes, cyclones, and violent gales occurring in some particularly exposed situations—give a maximum indicated velocity of 100 miles per hour. . . . Table 13 shows the maximum velocities observed at a number of stations by the United States Weather Bureau."*

TABLE 13.

Observatory.	Period.	Maximum Velocity Indicated.	Observatory.	Period.	Maximum Velocity Indicated.
Chicago, Ill.....	1871–1906	90	Savannah, Ga...	1894–1903	76
Buffalo, N. Y. ...	1871–1907	90	Philadelphia, Pa.	1872–1907	75
Galveston, Tex...	1894–1903	84	Bismarck, N. D..	1894–1903	72
New York, N. Y.	1871–1907	80	Boston, Mass...	1873–1907	72
Eastport, Me. ...	1873–1907	78	Salt Lake City, Utah.....	1894–1903	60

A tabulation, by months, of the highest indicated velocities recorded by the United States Weather Bureau, at the New York City Station, from 1884 to 1906, and of the number of different twelve-hour periods, during which a maximum velocity of 60 miles, or more, was observed, from 1895 to 1906, shows that:

The maximum velocity of 80 miles per hour occurred during a sleet-storm.

The maximum velocities occur during the winter months, when sleet may be on the wires.

Indicated velocities of more than 80 miles per hour will rarely, if ever, occur during the life of a given structure.

Indicated velocities of from 65 to 75 miles per hour may be expected several times each year, though much less frequently in conjunction with sleet.

In Table 14 are given the equivalent "actual" velocities corresponding to those "indicated" by anemometer readings, and the pressures per square foot produced on flat and cylindrical surfaces.

Experience in sleet-storms indicates that generally throughout this country a deposit of ice of about ½-inch thickness may be expected at irregular intervals. Greater thicknesses are sometimes encountered, but the heavier deposits are usually snow-ice of lighter weight, and with less adhesion to the wires. It may reasonably be expected that a portion, at least, of these larger accretions will be broken off by the rising wind, so that the final average load on a span

* "Overhead Construction for High-tension Electric Traction or Transmission," by R. D. Coombs, Trans. Am. Soc. C. E., vol. lx.

will be approximately equivalent to a uniform thickness of ½ inch of clear ice.

To a certain extent the thickness of ice is independent of the diameter of the wire, though it has sometimes been assumed that a thickness equal to the diameter would occur. This is manifestly wrong for the smaller sizes of wire, as is proved by the coating of twigs in every sleet-storm, and by actual experience with line wires.

TABLE 14.—WIND PRESSURES AND VELOCITIES.

Indicated Velocity, Miles Per Hour.	Actual Velocity, Miles Per Hour.	Pressure per sq. ft. on Cylinders. $P=.0025V^2$.	Pressure per sq. ft. on Flat Surfaces. $P=.0042V^2$.
30	25.7	1.7	2.8
40	33.3	2.8	4.6
50	40.8	4.2	7.0
60	48.0	5.8	9.7
70	55.2	7.6	12.8
80	62.2	9.7	16.2
90	69.2	12.0	20.1
100	76.2*	14.6	23.3
110	83.2*	17.3	29.1
120	90.2*	20.3	34.2

TABLE 15.—ICE AND WIND LOADS ON WIRES.†

COPPER WIRE—SOLID.

Gauge B. & S.	Breaking Strength.		Load Per Lin. Ft. Vertical.			Load Per. Lin. Ft. Horizontal.			Max. Load Per Lin. Ft. Plane of Resultant.		
	Hard-drawn.	Soft-drawn.	Dead.	Dead + ½″ Ice.	Dead + ¾″ Ice.	15.0 lbs. P. Sq. Ft.	8.0 lbs. P. Sq. Ft., on ½″ Ice.	11.0 lbs. P. Sq. Ft., on ¾″ Ice.	Dead, 15 lbs. Wind.	Dead, ½″ Ice, 8 lbs. Wind.	Dead, ¾″ Ice, 11 lbs. Wind.
0000.....	8310	5650	0.641	1.238	1.770	0.575	0.973	1.797	0.861	1.575	2.522
000.....	6590	4480	0.509	1.074	1.591	0.512	0.940	1.750	0.722	1.427	2.365
00.....	5220	3555	0.403	0.940	1.443	0.456	0.910	1.709	0.608	1.309	2.237
0.....	4560	2820	0.320	0.833	1.323	0.406	0.883	1.673	0.517	1.214	2.133
1.....	3740	2235	0.253	0.744	1.223	0.362	0.860	1.640	0.442	1.137	2.046
2.....	3120	1770	0.202	0.673	1.142	0.322	0.838	1.611	0.380	1.075	1.975
3.....	2480	1405	0.159	0.613	1.073	0.287	0.820	1.585	0.328	1.024	1.914
4.....	1960	1115	0.126	0.564	1.016	0.255	0.803	1.562	0.284	0.981	1.863
5.....	1560	885	0.100	0.524	0.969	0.227	0.788	1.542	0.248	0.946	1.821
6.....	1240	700	0.079	0.491	0.930	0.203	0.775	1.524	0.218	0.917	1.785

* Added by comparison.

† Abstract from wire tables, Fitzpatrick and Coombs, Engineers and Contractors, 1123 Broadway, New York.

TABLE 16.—ICE AND WIND LOADS ON WIRES.

COPPER WIRE—STRANDED.

Gauge B. & S.	Breaking Strength.		Load Per. Lin. Ft. Vertical.			Load Per. Lin. Ft. Horizontal.			Max. Load Per Lin. Ft. Plane of Resultant.		
	Hard-drawn.	Soft-drawn.	Dead.	Dead + ½″ Ice.	Dead + ¾″ Ice.	15.0 lbs. P. Sq. Ft.	8.0 lbs. P. Sq. Ft., on ½″ Ice.	11.0 lbs. P. Sq. Ft., on ¾″ Ice.	Dead, 15 lbs. Wind.	Dead, ½″ Ice, 8 lbs. Wind.	Dead, ¾″ Ice, 11 lbs. Wind.
500,000	23,540	13,340	1.525	2.345	2.989	1.024	1.213	2.126	1.837	2.640	3.668
450,000	21,210	12,020	1.373	2.163	2.791	0.963	1.180	2.081	1.677	2.464	3.481
400,000	18,860	10,680	1.220	1.984	2.599	0.910	1.152	2.042	1.522	2.294	3.305
350,000	16,500	9350	1.068	1.801	2.401	0.849	1.119	1.997	1.364	2.120	3.123
300,000	14,160	8025	0.915	1.618	2.203	0.788	1.087	1.953	1.208	1.949	2.944
250,000	11,790	6680	0.762	1.440	2.012	0.738	1.060	1.916	1.061	1.788	2.778
0000	9970	5650	0.645	1.286	1.831	1.663	1.020	1.861	0.925	1.641	2.611
000	7910	4480	0.513	1.116	1.651	0.588	0.980	1.806	0.780	1.485	2.446
00	6270	3555	0.406	0.978	1.498	0.525	0.947	1.760	0.664	1.361	2.311
0	4970	2820	0.322	0.866	1.372	0.469	0.917	1.719	0.569	1.261	2.199
1	3940	2235	0.255	0.771	1.263	0.413	0.887	1.678	0.485	1.175	2.100
2	3130	1770	0.203	0.695	1.174	0.364	0.861	1.642	0.417	1.107	2.019
3	2480	1405	0.160	0.633	1.103	0.326	0.841	1.614	0.363	1.053	1.955
4	1970	1115	0.127	0.582	1.042	0.289	0.821	1.587	0.316	1.006	1.899
5	1560	885	0.101	0.540	0.992	0.258	0.804	1.564	0.277	0.970	1.852
6	1235	700	0.080	0.505	0.951	0.230	0.789	1.543	0.243	0.936	1.813

TABLE 17.—ICE AND WIND LOADS ON WIRES.

COPPER WIRE—SOLID, TRIPLE BRAID WEATHER-PROOFING.

Gauge B. & S.	Breaking Strength.		Load Per. Lin. Ft. Vertical.			Load Per. Lin. Ft. Horizontal.			Max. Load Per. Lin. Ft. Plane of Resultant.		
	Hard-drawn.	Soft-drawn.	Dead.	Dead + ½″ Ice.	Dead + ¾″ Ice.	15.0 lbs. P. Sq. Ft.	8.0 lbs. P. Sq. Ft., on ½″ Ice.	11.0 lbs. P. Sq. Ft., on ¾″ Ice.	Dead, 15 lbs. Wind.	Dead, ½″ Ice, 8 lbs. Wind.	Dead, ¾″ Ice. 11 lbs. Wind.
0000	8310	5650	0.767	1.476	2.064	0.800	1.093	1.961	1.108	1.837	2.847
000	6590	4480	0.629	1.309	1.882	0.741	1.062	1.918	0.972	1.686	2.687
00	5220	3555	0.502	1.133	1.682	0.644	1.010	1.847	0.818	1.518	2.498
0	4560	2820	0.407	1.029	1.573	0.625	1.000	1.833	0.746	1.434	2.415
1	3740	2235	0.316	0.909	1.438	0.564	0.968	1.790	0.646	1.328	2.296
2	3120	1770	0.260	0.843	1.367	0.546	0.958	1.775	0.605	1.276	2.240
3	2480	1405	0.199	0.763	1.278	0.507	0.937	1.747	0.545	1.208	2.164
4	1960	1115	0.164	0.698	1.199	0.449	0.906	1.704	0.478	1.143	2.083
5	1560	885	0.135	0.660	1.146	0.430	0.896	1.690	0.451	1.113	2.042
6	1240	700	0.112	0.627	1.118	0.410	0.885	1.675	0.425	1.084	2.014

Inasmuch as the ice and wind loads are both acting under maximum loading, some reasonable combination must be assumed. Since the surface exposed to wind pressure is the diameter of the ice-covered wire, if no ice load is present it would be necessary to assume an extremely high wind velocity to obtain a maximum load equivalent to a moderate combined load.

In Tables 15, 16, and 17 are given the breaking strengths, and the loads per lineal foot of wire, for various sizes of wire and conditions of loading.

TABLE 18.—TELEPHONE AND TELEGRAPH WIRES.

HARD-DRAWN, SOLID, BARE COPPER.

Gauge.	Diam.	Area.	Ultimate Strength.	Load Per lin. Foot. Vertical.			Load Per Lin. Ft. Horizontal.			Max. Load Per Lin. Ft. Plane of Resultant.		
				Dead	Dead + ½″ Ice	Dead + ¾″ Ice	8.0 lbs. Wind	8.0 lbs. P. Sq. Ft., on ½″ Ice	8.0 lbs. P. Sq. Ft., on ¾″ Ice	Dead, 8 lbs. Wind	Dead, ½″ Ice, 8 lbs. Wind	Dead, ¾″ Ice, 8 lbs. Wind
No. 8 B. W. G.	0.165	0.0214	1285	0.082	0.496	0.936	0.110	0.777	1.110	0.137	0.922	1.452
No. 9 B. & S.	0.114	0.0103	620	0.039	0.421	0.845	0.076	0.743	1.076	0.085	0.854	1.386
No. 12 N. B. S.	0.104	....	....	0.033	....	....	....	....	....	....	....	....
No. 14 N. B. S.	0.080	....	....	0.019	....	....	....	....	....	....	....	....
P. R. R. (old.)	0.110	0.0095	570	0.037	0.416	0.802	0.073	0.740	1.073	0.082	0.849	1.339

The transition in pole capacity from a telegraph or telephone line to a power transmission line is not necessarily discernible; in fact, the load per foot of line may be greater in the former case. In telephone and telegraph practice it is desirable that the wire spacing and the sag in the wires be kept as small as possible. This requirement immediately places a limit upon the length of span, since the wires require support at frequent intervals. In power transmission, however, the length of span may be much greater, as it is not necessary to place the wires close together and the sags may be increased. Economically speaking, the span should be as great as the proper spacing of wires and the necessary clearance between the wires and the ground will permit. Within limits, an increase in the span length merely adds an inappreciable amount of wire, requires a greater distance between the wires, a slight increase in the pole height, and, sometimes, a better grade of insulator. On the other hand, both the number of insulators and the number of poles is reduced.

While it is common practice to use wire guys, in open country, both normal and parallel to the line, in populated districts the use

of guys is necessarily restricted, and in any case constitutes a nuisance and expense in maintenance.

While the combination of a high wind and a large accretion of ice is not entirely unknown, such combinations are not very frequent, occurring perhaps once in a decade. If it is desired to reduce the first cost as much as is compatible with safety, the poles may be designed, using high-unit stresses for conditions that rarely, if ever, occur. In regard to the factors of safety, unit stresses, and working stresses, to be allowed in the constituent materials of a reinforced concrete pole, there is as much room for latitude of judgment as in other structural work. The character of service is not closely akin to that of bridges or buildings, and the factors of safety common to such work would be somewhat conservative, for poles computed for extreme conditions of loading.

The present practice differs rather widely as to the most economical or most desirable distribution of reinforcement. It is now generally conceded, in reinforced concrete work, that the finer the distribution of metal, the greater the homogeneity and strength of the construction. However, in the case of poles, where the concrete is deposited within narrow forms, other conditions partly modify or control the distribution. If the metal is concentrated in four equal areas, a rod to each corner, a square pole will be equally strong, either parallel or normal to the line. Other or finer distribution of metal with equal strength in both directions necessitates an excess of material over that required for the forces normal to the line. When the metal is concentrated, the fabrication of the reinforcement into a unit frame, and also the concreting operations, are more easily accomplished. It may be said, as in the case of beams, that ample web reinforcement assures a firm unyielding unit during concreting, as well as provision against vertical shearing stresses.

In other fields of reinforced concrete work high-carbon steel with a high elastic limit, and a correspondingly richer concrete, are being used, permitting higher working stresses in design. If, in such work, high-unit stresses can be used, with a large percentage for impact, it would seem entirely reasonable to use correspondingly high working stresses in pole design, since the severe conditions of loading occur infrequently.

The most commonly used mixture is 1 : 2 : 4 Portland cement, sand, and broken stone or gravel. It should be mixed wet, using carefully selected materials, with the fine aggregate next to the forms, and tamped or churned to eliminate air-bubbles. Such a

mixture has an average compressive strength of about 900 pounds per square inch in seven days, 2400 pounds per square inch in one month, 3100 pounds per square inch in three months and 4400 pounds per square inch in six months. If conditions make it desirable to use high working stresses, a month or more should elapse before new poles undergo severe tests.

Since in solid poles of light capacity the loading produces a low compressive unit stress in the concrete, a considerable area of concrete might be omitted, or, theoretically, the economical section would be a hollow one.

The increased weight of a solid pole renders it more difficult to handle, and a hollow pole would therefore be more economical in erection. Further, the sides of the pole resisting the bending stress normal to the line might be at a greater distance from the center than the sides perpendicular to the line.

There are, however, certain objections to the use of hollow or unsymmetrical sections. The former are difficult to make properly, and the cost of the forms is greatly in excess of that required for solid sections. The unsymmetrical sections may perhaps be open to criticism on the score of appearance, and if the lack of symmetry is very pronounced, render the poles relatively weak in the direction of the line. Conservative reasoning would dictate that such poles, sometimes styled "whip lash" construction, should be interspersed with "dead-end" poles of heavier design.

In general, a square, octagonal, circular or other cross-section may be used, but it is desirable as a matter of appearance, since sharp corners are difficult to make and subject to accident, that all such corners be chamfered or rounded. The minimum diameter, or width, at the top may be made 6 inches for small poles, and increased as required for the strength and appearance of long poles or poles carrying a heavy line. In any case care should be exercised, in determining the taper and reinforcement, that no weak section occurs at some distance above the ground-level.

As a study of the size and appearance of reinforced concrete poles of different strengths, and to illustrate the relative line capacities represented by the various loadings given in the preceding experiments, a number of designs were made, which are shown in Figs. 32, 32A, 33, 33A, 34, and 34A. Three different line capacities were assumed and two designs made for each, in order to show the influence of high-unit stresses on the appearance, weight, and cost of the poles.

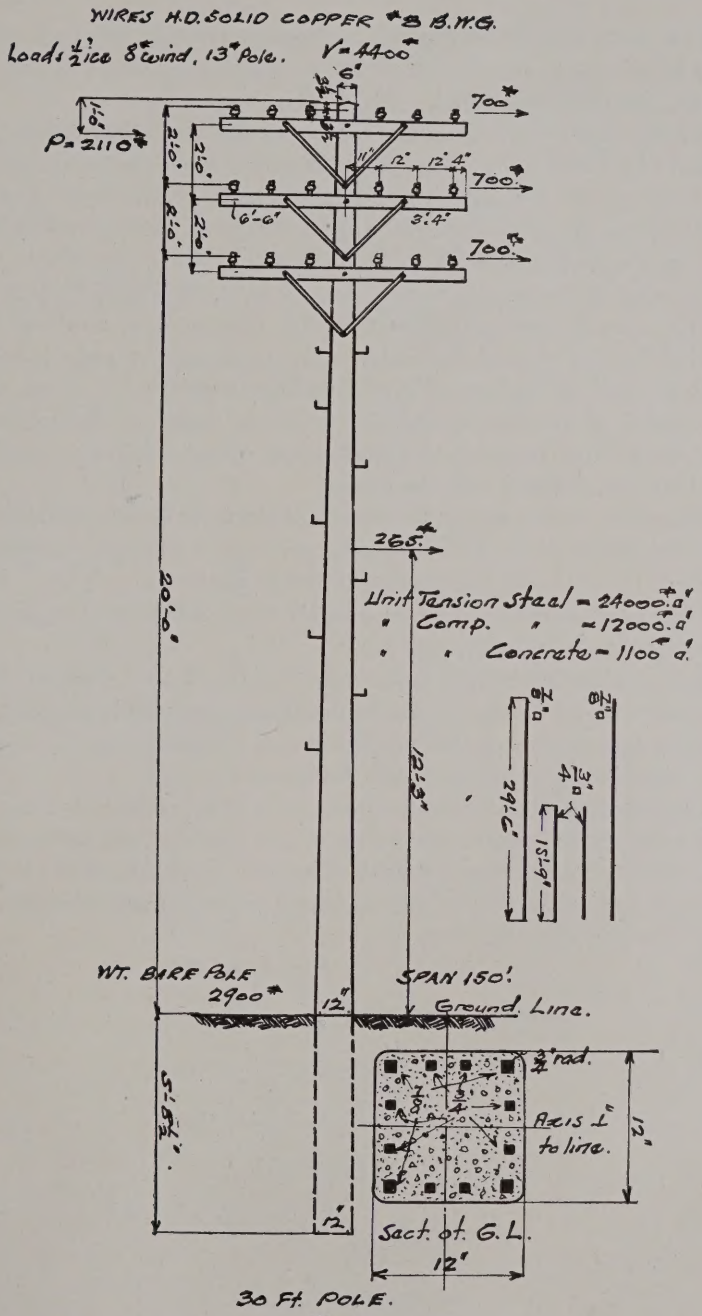

WIRES H.D. SOLID COPPER #8 B.W.G.
Load: ½" ice & wind, 13# Pole. V=1400#
6'
1'-0"
P=2110#
2'-0"
2'-0"
2'-0"
2'-0"
700#
700#
700#
11"
12"
12'-4"
6'-6"
3'-4"
265#
20'-0"
Unit Tension Steel = 24000 #/□"
" Comp. " = 12000 #/□"
" " Concrete = 1100 #/□"
12'-3"
27'-0"
15'-9"
WT. BARE POLE 2900#
12"
SPAN 150'
Ground Line.
5'-6½"
¾" rad.
Axis ⊥ to line.
12"
12"
Sect. of G.L.
12"
30 Ft. POLE.

Fig. 32.

The poles shown in Fig. 32 were designed for 150-foot spans, those in Figs. 33 and 33A for 120-foot spans, and those in Figs. 34 and 34A, for 100-foot spans.

All the poles were designed for the same elemental loads, *i. e.*, ½ inch of ice and 8.0 pounds per square foot wind pressure thereon, and 13.0 pounds per square foot wind pressure on the pole; and the loads equivalent to ½ inch of ice and 2.0 pounds wind pressure have been shown on Figs. 32A, 33A, and 34A merely for comparison.

The unit stresses, quantities, etc., are given in Table 19, page 56.

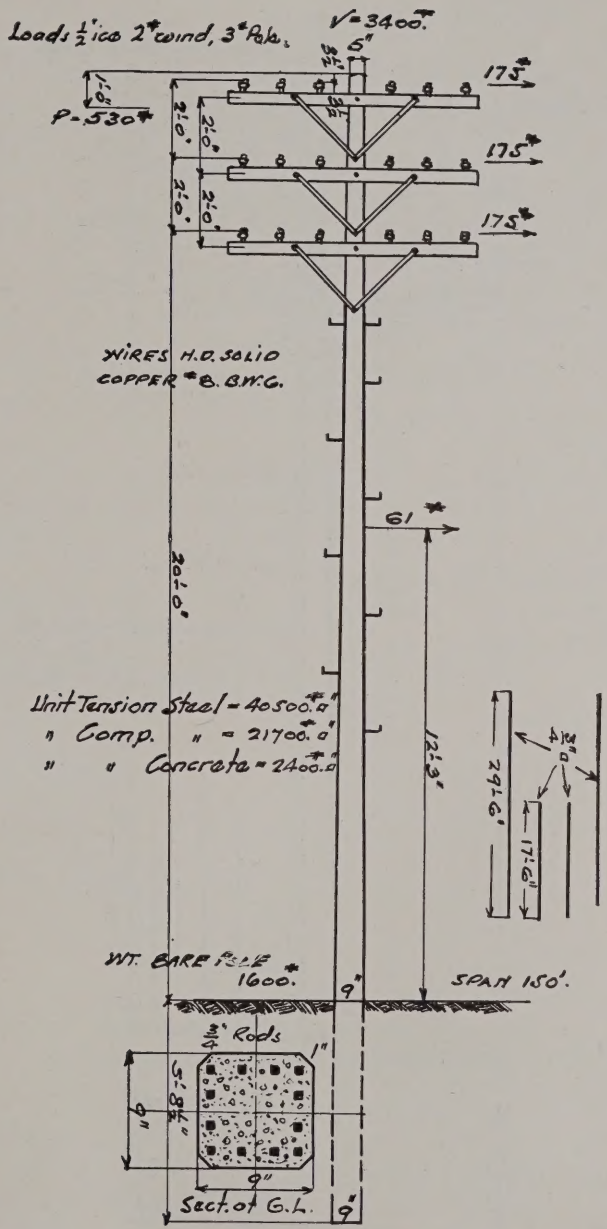
The web system, not shown in the illustrations, consists in a spiral of No. 12 wire placed outside the rods and securely attached to them, and in horizontal ties 1″ wide × ⅛″ to ¼″ thick, 3 to 5 ft. apart, depending upon the particular design. The reinforcement thus forms an independent skeleton, which can be assembled, handled, and lowered into the forms.

The poles were designed to withstand the total force perpendicular to the direction of the line, and good practice apparently justifies an equal, or nearly equal, strength parallel to the line.

The linear variation of stress, and the usual methods of computing beams doubly reinforced, were used.

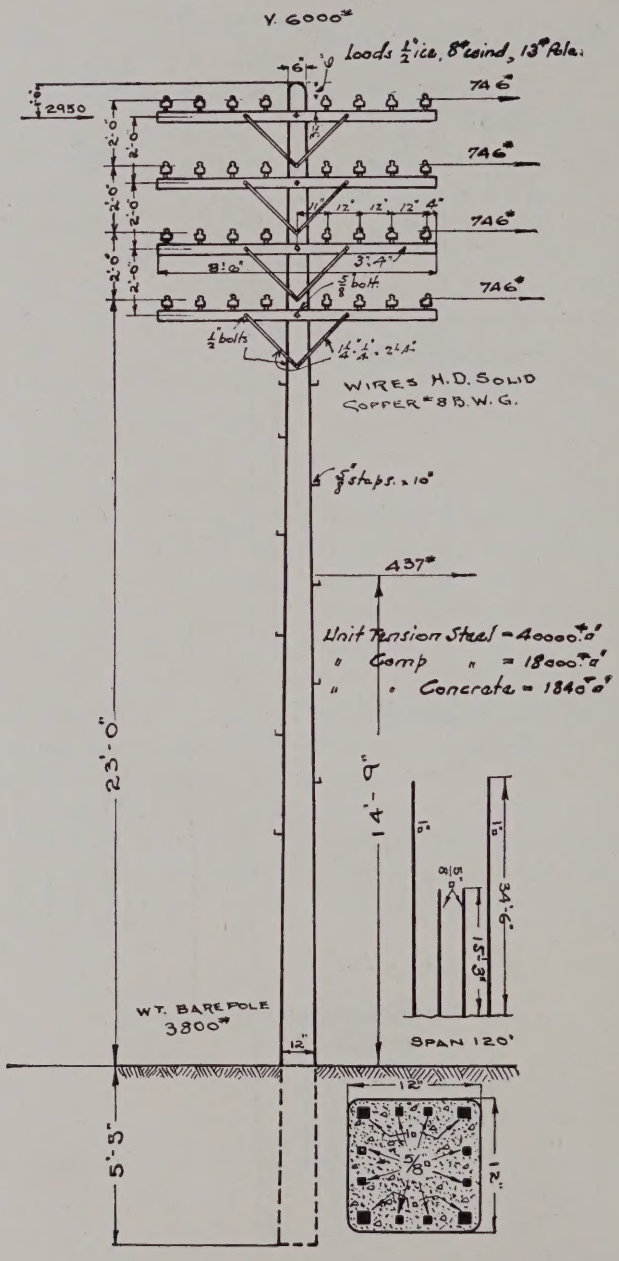
The reinforcing rods were assumed to be of a mechanical bond type, particularly in the designs with high-tension steel, although the frequent attachment of the web system would doubtless be of assistance in developing the necessary bond.

The extreme fiber stress in concrete in compression in these designs is about the one-month value of the crushing strength of concrete, and it is more than probable that such poles will have attained a greater age before they would be tested under the actual conditions for which they were figured.



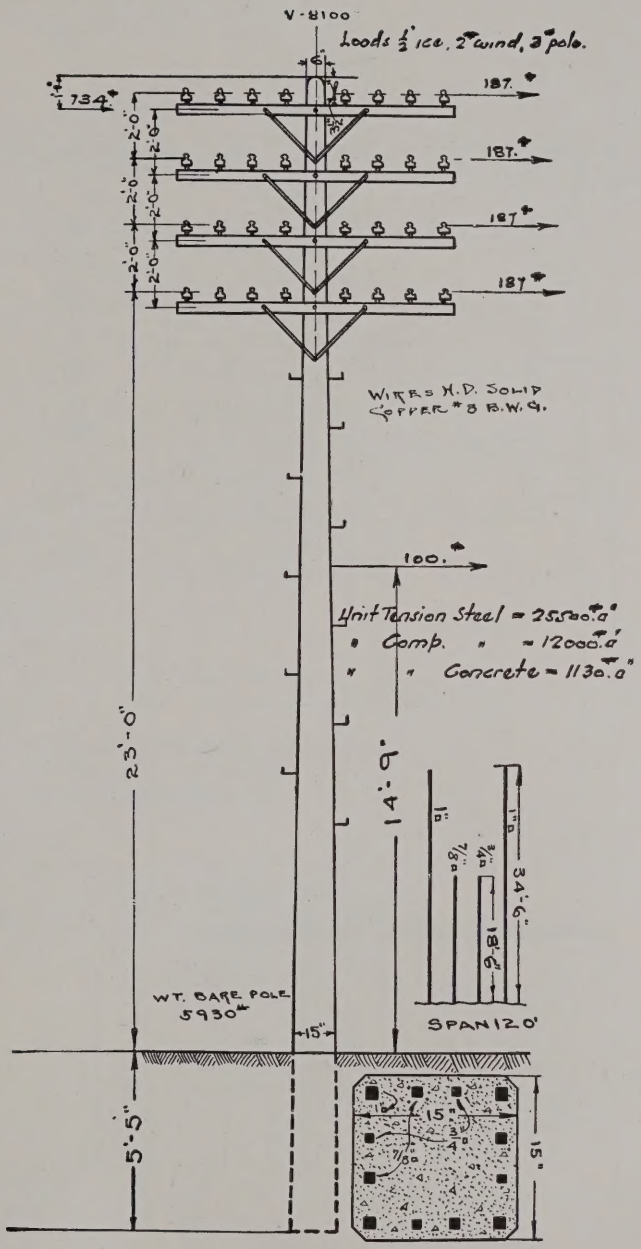
30 Ft. POLE.

Fig. 32A.



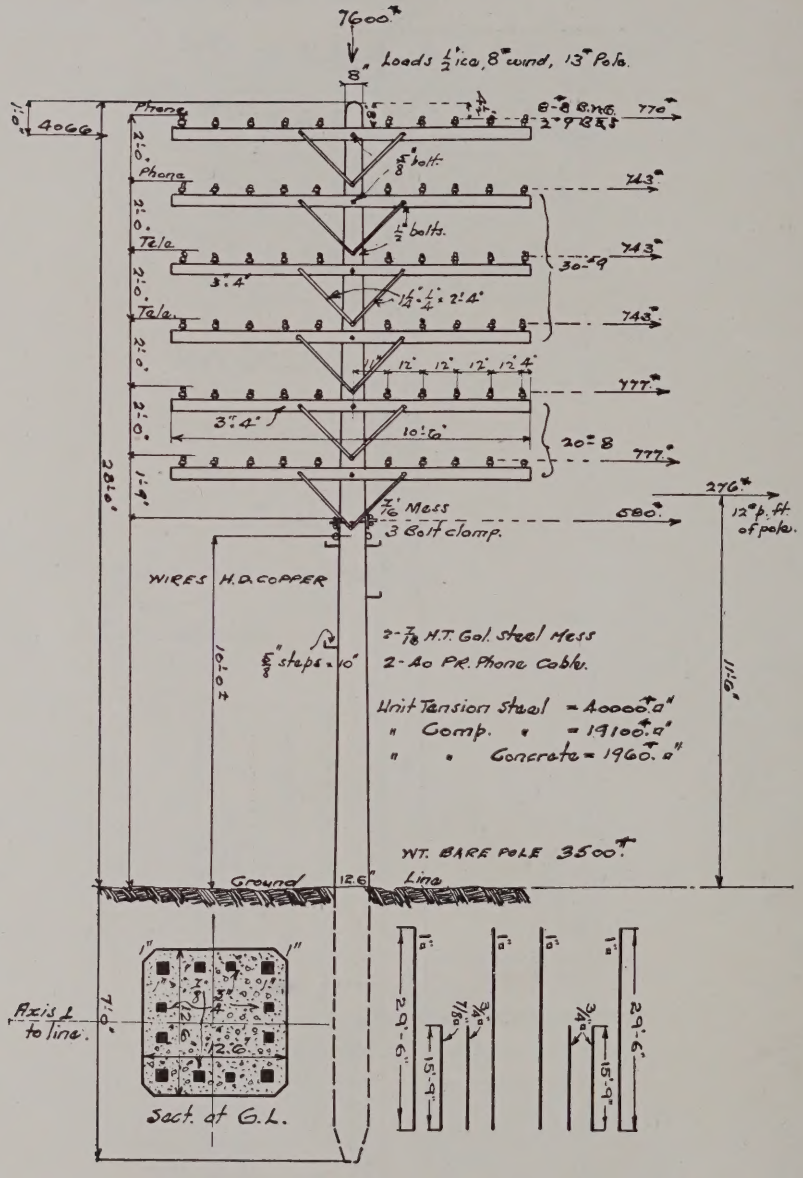
35'-0" POLE

Fig. 33.



35'-0" POLE.

Fig. 33A.



Typical Trunk Line Pole.

Fig. 34.

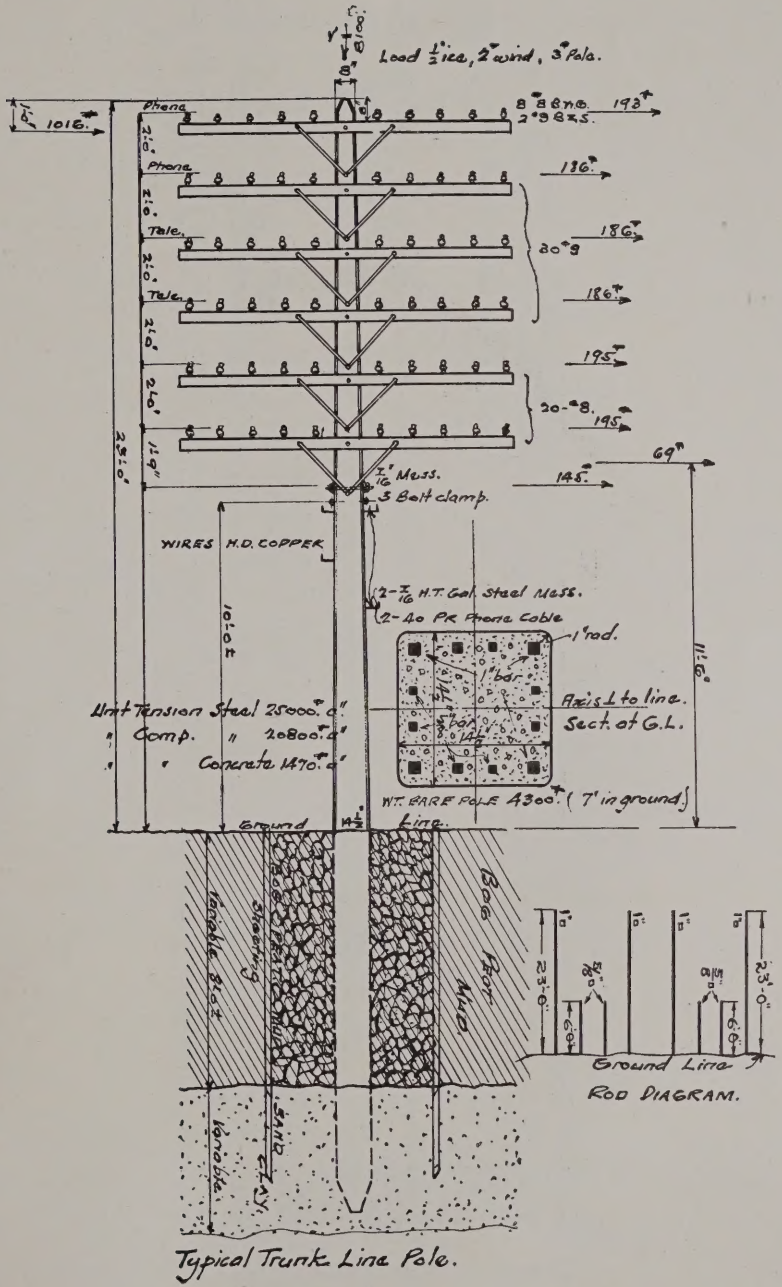

Load ½" ice, 2" wind, 3 Pole.
Phone
Phone
Tele.
Tele.
8 #8 Bare 193
2 #3 E.B.B.
136
186
186
195
195
69
145
3 Bolt clamp.
WIRES H.D. COPPER
2-40 PR Phone Cable
1" rod.
1" bar
Axis ⊥ to line.
Sect. at G.L.
WT. BARE POLE 4300 (7' in ground)
Ground Line
Unit Tension Steel 25000
Comp. 20800
Concrete 1470
Loose Earth Filling
Solid Rock
Loose Earth Filling
Solid Rock
Ground Line
ROD DIAGRAM.
Typical Trunk Line Pole.

Fig. 34A.

TABLE 19.—UNIT STRESSES AND QUANTITIES.

Pole.	Total Bal. Foot Lbs.	Width at Ground L.	Span	Edge Dist. for Steel.	Req'd A. for Steel.	Unit Ten. Stress S.	Unit Comp. Stress Steel.	Gross % Steel G. L.	% Tension Steel G. L.	Ext. Fibre Stress Concrete.	Cu. Ft. Concrete.	Wt. Steel Rods.	Webbing.	
													St. Spiral.	Lfc. Ties.
Trunk	(22400) 89460	12.6 in	100′–0″	1.75 in.	3.38 sq. in.	40000 ″	19100 ″	5.60%	2.40%	1960 ″	23.0	660	270	47
Do	89460	14.5 in	do do	1.50 in.	2.72 “ “	25000 ″	20800 ″	3.38%	1.44%	1470 ″	28.0	540	280	50
Light 30 ft.	(12570) 49450	12.0 in	150′–0″	1.25 in.	2.64 “ “	24000 ″	12000 ″	5.25%	2.04%	1100 ″	19.0	550	300	55
Do	49450	9.0 in	do do	1.25 in.	2.25 “ “	40500 ″	21700 ″	3.33%	3.22%	2400 ″	10.0	493	260	45
Medium 35 ft.	21000 84034	12.0 in	120′–0″	1.50 in.	2.78 “ “	40000 ″	18000 ″	4.95%	2.20%	1840 ″	39.0	632	320	60
Do	94034	15.0 in	do do	1.50 in.	3.32 “ “	25500 ″	12000 ″	4.12%	1.64%	1130 ″	25.0	300	330	65

Note: 2″ ice, 8 lbs. per sq. ft. wind on wires, and 13 lbs. per 50 ft. wind on pole, wind velocity 70 miles per hour.

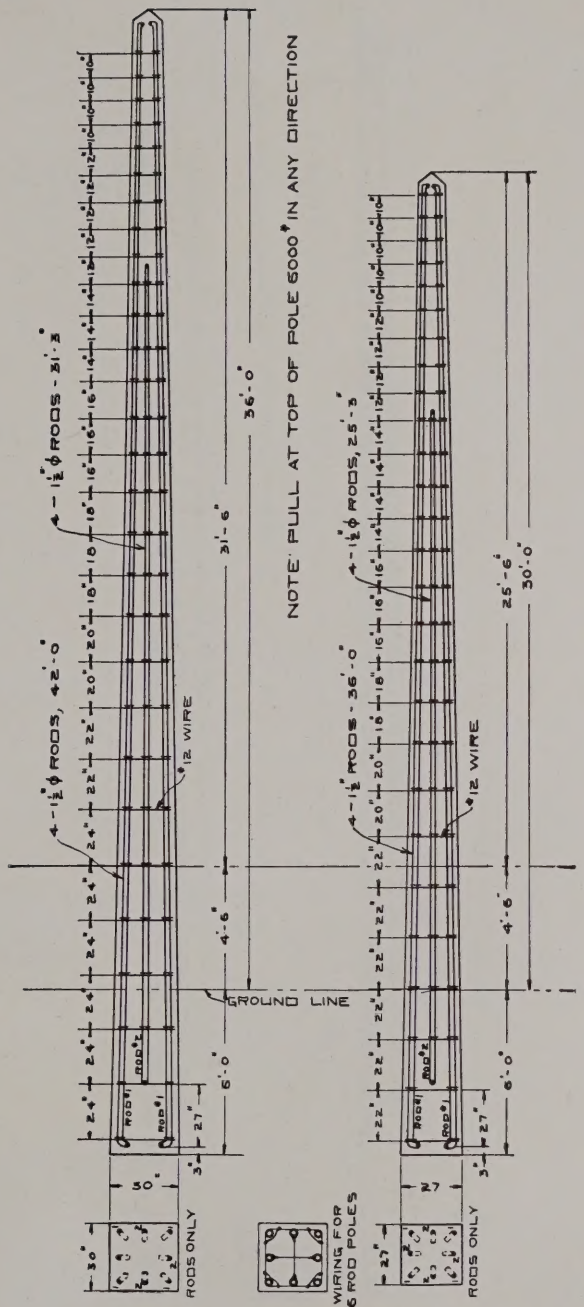


Fig. 35.—Hennebique pole design.

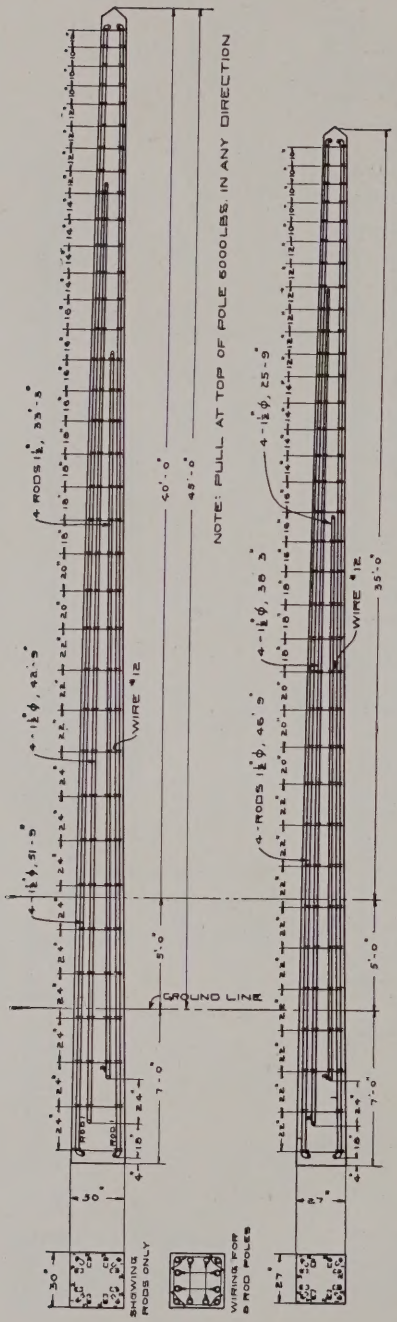


Fig. 36.—Hennebique pole design.

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(Continued from Back Cover)

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Standard Portland Cement Corporation
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Continued on Third Cover Page